STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION

DRAINAGE HANDBOOK
STORMWATER MANAGEMENT FACILITY

OFFICE OF DESIGN, DRAINAGE SECTION January 2004
TALLAHASSEE, FLORIDA

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Chapter 1

Introduction

1.1 Background

The 1987 Florida Department of Transportation Drainage Manual was published as a three volume set: Volume 1 - Policy; Volumes 2A and 2B - Procedures; Volume 3 - Theory. On October 1, 1992, Volume 1 - Policy was revised to Volume 1 - Standards. With that revision, Volumes 2A, 2B, and 3 were designated as general reference documents. The Volume 1 - Standards was revised in January 1997 and was renamed to simply the “Drainage Manual.” No revisions have been, nor will be made to volumes 2A, 2B, and 3 of the 1987 Drainage Manual.

This handbook is one of several the Central Office Drainage section is planning to develop to replace Volumes 2A, 2B, and 3 of the 1987 Drainage Manual. In this form, the current Drainage Manual will be maintained as a “standards” document, while the handbooks will cover general guidance on FDOT drainage design practice, analysis and computational methods, design aids, and other reference material.

1.2 Purpose

This handbook is intended to be a reference for designers of projects in the project development phase or the design phase. Guidelines are provided to help designers select stormwater[1] management facility sites, to address concerns that have existed with operations of the Department’s stormwater facilities. Guidelines are also provided for the calculations associated with stormwater management facilities. Pertinent sections of the 1987 Drainage Manual have been incorporated into this handbook.

The guidance and values provided in this handbook are suggested or preferred approaches and values, not requirements nor standards. The values provided in the Drainage Manual are the minimum standards. In cases of discrepancy, the Drainage Manual standards shall apply. As the Drainage Manual states about the standards contained in it, situations exist where the guidance provided in this handbook will not apply. THE INAPPROPRIATE USE OF AND ADHERENCE TO THE GUIDELINES CONTAINED HEREIN, DOES NOT EXEMPT THE ENGINEER FROM THE PROFESSIONAL RESPONSIBILITY OF DEVELOPING AN APPROPRIATE DESIGN.

This handbook should be useful to engineers relatively new to the field of designing FDOT stormwater management facilities.

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[1] Your trivia for the day! The State of Florida refers to "stormwater" as one word. The Federal Government refers to "storm water" as two words.

1.3 Regulatory Framework

The Department operates under Federal, State, and Local laws and regulations. Chapter 5 of the Drainage Manual lists specific regulation relating to stormwater management. Appendix C of the Drainage Manual describes the regulations in more detail.

1.4 General Discussion of This Handbook

1.4.1 Units

Metric and US customary units have been used in this handbook. Most discussions and examples use both units, with metric units given first followed by US customary units in parentheses. Some discussion and parts of some examples are in US customary units only, for example rainfall depths.

1.4.2 Examples

Within some examples, you will find text in shadow boxes. This is intended to be guidance applicable to situations or projects, which are outside the scope of the example. The examples are listed in the Table of Contents.

1.4.3 Possible Plan Notes

There are several suggested plan notes in the handbook. These are provided to promote standard wording for notes that may be commonly used on the Department's projects. A particular note may not apply to your project. Even where a note is applicable, its wording may need to be different from that in the handbook. It is up to the responsible engineer to determine if the notes are appropriate for a project. The notes are listed below with page reference.

1.4.4 Distribution

This handbook is available for downloading from the Drainage Internet site.

1.4.5 Revisions

Any comments or suggestions concerning the handbook can be made by mailing them to:

Florida Department of Transportation
Office of Design - Drainage Section
Mail Station 32
605 Suwannee Street
Tallahassee, FL 32399-0450

1.5 Definitions of Terms and Acronyms

Attenuation	To temporarily hold back or store stormwater to control the rate of discharge. Normally the term is associated with flood control. Also, see Detention.
Critical Duration	As defined by Rule 14-86 F.A.C.: "Critical Duration" means the duration of a specific storm event (frequency, i.e., 100 year) which creates the largest volume or highest rate of net stormwater runoff (post-development runoff less pre-development runoff) for typical durations up through and including the 10-day duration event. The critical duration is determined by comparing various durations of the specified storm and calculating the peak rate and volume of runoff from each. The duration resulting in the highest peak rate or largest total volume is the "critical duration" storm. "See the beginning of Chapter 5 for interpretation."
FDEP	Florida Department of Environmental Protection
FDOT	Florida Department of Transportation
FHWA	Federal Highway Administration
Department	Florida Department of Transportation

Detention	To temporarily hold back or store stormwater to control the rate of discharge. Normally the term is associated with water quality control such as wet detention. Sometimes the term is used for flood control attenuation.
Infiltration Rate	The maximum rate at which water can enter the soil from the surface under specified conditions. The units are length per time.
NRCS	National Resource Conservation Service (formerly Soil Conservation Service)
NWFWMD	Northwest Florida Water Management District
Positive Outlet	As defined by Rule 14-86 F.A.C.: A point of stormwater discharge into surface waters which under normal conditions would drain by gravity through surface waters ultimately to the Gulf of Mexico, or the Atlantic Ocean, or into sinks or closed lakes provided the receiving water body has been identified by the appropriate Water Management District as functioning as if it recovered from runoff by means other than transpiration, evaporation, percolation or infiltration.
Recovery Time	The time it takes to infiltrate the retention volume.
Retention	To retain stormwater and prevent any surface water discharge. The retained stormwater is either infiltrated into the ground or evaporated.
Retention Volume *Not applicable to wet detention facilities.*	The volume between the pond bottom and the lowest discharge elevation of the outlet control structure. For retention systems discharging to open basins, the retention volume is the treatment volume. For retention systems discharging to closed basins, the retention volume is usually the volume, which must be retained to ensure that the post developed discharge volume does not exceed the pre-developed discharge volume.

SFWMD	South Florida Water Management District
SHWT	Seasonal High Water Table (Discussed in Chapter 4)
SJRWMD	St. Johns River Water Management District
SRWMD	Suwannee River Water Management District
SWFWMD	Southwest Florida Water Management District
Treatment	Generally referring to stormwater management practices to improve the quality of stormwater discharged.
Treatment Volume	The volume of runoff usually associated with the first flush, which must be retained, detained, or filtered to remove pollutants and improve water quality.
WMD	Water Management District

Chapter 2

Selecting a Pond Site

Selecting the most appropriate pond site requires the work of many different offices and professionals within the Department. You, as a drainage designer, will provide critical information, but because of the many factors to consider, a team approach is recommended.

There are numerous design features (depth, size, shape, treatment method, etc.), which you can modify to accommodate a pond site. However, hydraulic constraints may preclude the use of some sites. Alternate sites and their different design features will usually result in different costs and impacts. As a result, an evaluation of alternates must be made to select the most appropriate pond site. The purpose of the evaluation is two fold. First, it will show that alternate sites were considered and that the selected site was the most appropriate. Second, when you combine the evaluation with the final design details, they become the documentation that justifies the need to acquire property rights.

The evaluation[2] should weigh and balance numerous factors such as cost, maintainability, constructability, public opinion, aesthetics, and environmental, social, and cultural impacts. The costs consisting of right of way, environmental, construction, and long term maintenance are usually the easiest factors to estimate and compare. Other factors are more subjective and qualitative. Because a broad range of subjects is involved, a multi functional team approach is strongly recommended to select the most appropriate pond site. Teams should have representatives from right of way, design, drainage, environmental management, maintenance, construction, and eminent domain. At times other units may provide critical information to the evaluation process. Although all of the team members may not participate in the entire process, they will likely provide critical information at some stage. The project manager, with support from the Drainage and Right of Way offices, will be responsible for coordinating the team effort and ensuring that the appropriate personnel participate.

Pond site evaluations must be done in the Project Development Phase. Often pond sites are reevaluated during the Design Phase. Before doing a "design reevaluation," check what commitments have been made and what work has been done during the Project Development Phase. Design reevaluations must be complete before the Phase I plans submittal.

[2] Where one person owns all the property in the area and that person is agreeable to any location proposed by the Department, evaluating alternates may not seem necessary. In these situations, the evaluation will not be as extensive as in other situations, nevertheless, some evaluation should be done to show that the site selected results in the lowest total cost.

CONSIDERATIONS WHEN SELECTING A POND SITE

1. Use existing FDOT properties or other state owned property, if feasible.

2. Minimize the number of parcels required. For example, avoid using part of two parcels when the pond will fit within one.

3. Generally, property owners prefer to have ponds placed toward the rear of their property. For parcels that abut the roadway right of way, the portion of the parcel next to the road is usually the most expensive.

4. Avoid splitting a parcel, thus creating two independent parcel remainders.

5. Consider the parcels identified by the right of way office. Even if a parcel is not large enough to provide all the stormwater management, it may be large enough to provide the treatment for stormwater quality. Or it could replace treatment and attenuation for parcels adjacent to the road that will have their ponds removed because of the road improvements

6. Avoid wetlands.

7. Avoid archaeological sites and historic structures listed on or eligible for listing on the National Register of Historic Places.

8. Consider a joint use facility (one the Department and another entity share) as an alternate, if one is feasible.

9. Generally, do not consider an option that requires water quality monitoring. Historically this has been very expensive.

10. Stormwater treatment systems must be at least 30 meters (100 feet) from any public water supply well. (Chapter 62-555, F.A.C.)

11. Locations with billboards are usually expensive.

2.1 Estimating Right of Way Requirements

The right of way required for a pond site varies with the amount of additional impervious area and associated additional runoff, the ground line and groundwater elevations at the pond, the proposed road elevations, and sometimes the soil conditions and other factors. During the pond site evaluation stage, the accuracy to which you estimate these items and the resulting pond size varies with several factors. The most important factor is the schedule for acquisition of the pond site.

Sometimes the acquisition schedule dictates that results of the pond site evaluation form the basis for the final pond site right of way requirements. For these projects, you should determine the pond size as accurately as if doing the final detailed design.

There are other projects where the determination of the final right of way requirements occurs shortly after the pond site evaluation. The acquisition process starts after the final right of way requirements are established. For these projects, the pond site evaluation is done only to compare alternate sites or drainage schemes and your size estimates should be accurate enough to minimize changes to the right of way requirements during the final design.

There is a third category of projects where the right of way acquisition is scheduled several years after the pond site evaluation or the acquisition is not even funded in the Department's work program. For these projects, changes in pond size and location from that established in the original evaluation will not substantially effect production schedules nor the right of way acquisition process. Therefore, your pond size estimates need not be very accurate. For these projects, a pond site reevaluation is typically done shortly before right of way acquisition.

Other factors that affect the level of accuracy for pond size estimates are property costs and the existing and anticipated development of the project area. In a rural area with relatively large tracts of land, changes to pond size and location will have less impact to property owners and the Department, than in an expensive urban area that is rapidly developing and has relatively small parcels. As a result the pond size estimates used for these evaluations do not need to be as accurate as in urban, rapidly developing areas.

2.1.1 Typical Factors Controlling Surface Area Requirements

The surface area requirements for a pond are typically dictated by the need to fit storage volumes within upper and lower constraints. Sometimes the requirement to recover retention storage volume in a prescribed time dictates the size of the pond. The following items could control the surface area requirements for a pond.

- The top of the treatment and attenuation volume are constrained to the ground line at the pond (or the berm elevation) minus the freeboard.

- For urban projects, the hydraulic gradient of the storm drain is constrained to the low point in the gutter minus the hydraulic gradient clearance. This constraint is often critical in flat terrain but not steep terrain.

- The previous three items are constrained on the bottom to groundwater elevations or sometimes discharge tailwater elevations. The groundwater elevation constraint will vary with the method of treatment used and the requirements of the regulatory agency.

- Retention ponds must recover a certain volume in a certain time. The size of the pond bottom area sometimes controls the recovery or drawdown time. This may be particularly critical for ponds discharging to closed basins.

- For wet detention facilities, the treatment volume is limited by most regulatory agencies to 0.45 meters (18 inches) and the required permanent pool volume must be provided.

- In rolling or steep terrain, the low side of the site is often bermed to contain a substantial portion of the pond volume. The horizontal distance of the embankment from the berm top to natural ground dictates how much right of way is required in this direction. The embankment slope must be flat enough to be stable. For example, a 1(vert) to 2(horiz) slope in sandy soil with seepage may not be stable. A slope stability analysis may be appropriate. Discuss these situations with the geotechnical engineer to establish an acceptable slope and thus a reasonable estimate of the surface area requirements.

Example 2.1 Estimating Pond Right of Way Requirements

Given:

- Flat Terrain, approx. 1% slope
- Proposed Pond Discharges to Open Basin
- Proposed Curb & Gutter Section with Gutter Elevation at the Low Point in Profile = 18.25 m
- Ground Elevation at Pond Site = approx. 18 m
- Est. SHWT = 0.75 m - 1.1 m below ground (based on NRCS soil survey)
- Treatment Volume = 310 m^3
- Est. Peak Attenuation Volume = 554 m^3 (from Example 5.1)
- Est. 3 - year Attenuation Volume = 290 m^3 (storm drain design frequency)

Find: Estimated Surface Area Requirements for a Pond

1. A wet detention pond is used since the SHWT is so close to the surface.

For these conditions, the surface area is typically controlled by one of two requirements. Both involve spreading the treatment and attenuation volumes over a large enough area to keep the height of the volume within constraints. The height (H) of the treatment and peak attenuation volume is constrained on the top by the ground elevation minus the freeboard, and on the bottom by the controlling groundwater elevation. Although some WMD's allow treatment below SHWT, this example will assume that treatment is above the SHWT. First, determine the surface area necessary to meet these constraints. The other requirement that may control the surface area is discussed after step 5.

2. Conservatively assume the SHWT is 0.75 meters below ground. The standard freeboard is given in the Drainage Manual. The treatment and peak attenuation volume are constrained to the following height(H).

   H = Depth to SHWT – Freeboard
   H = 0.75 - 0.3
   H = 0.45 m

3. The total peak storage volume required is

   $Volume_{PEAK}$ = Treatment Volume + Est. Peak Attenuation Volume
   $Volume_{PEAK} = 310 + 554 = 864\ m^3$

You will need to make assumptions about the pond configuration.

Shape: Assume it will be rectangular. Irregular shapes can usually be approximated by a rectangular shape so this is a reasonable assumption and it greatly simplifies estimating the surface area.

Length to Width Ratio (L/W): The property lines may suggest a preferred ratio to make best use of a parcel. Without other guidance, assume L/W = 2.

Side Slopes: Assume flat slopes such as 1(vertical) to 5 or 6 (horizontal) for sites required to be aesthetically pleasing. Assume 1(vertical) to 4(horizontal) for most other conditions.

4. Use the formula for a rectangular box to determine the water surface area of a pond with vertical sides.

Volume = L_{RECT} W_{RECT} H
where: H= height (m) = 0.45 m for the above condition
L_{RECT} = length (m) of vertical sided pond
W_{RECT} = width (m) of vertical sided pond

Assume for this example that L / W = 2, then

864 m^3 = L_{RECT} x (0.5 L_{RECT}) x 0.45 m, then
L_{RECT} = 62.0 m
W_{RECT} = 31.0 m

5. Increase these dimensions to account for sloped sides by adding: 2 x (0.5 x H x Side Slope).

For this example assume side slope = 5, thus adding 2.3 m to each dimension.

Length @ top of slope = 64.3 m
Width @ top of slope = 33.3 m

Then,
Water Surface at Peak Design Stage = 64.3 x 33.3 = 2141 m^2 = 0.21 ha

The other requirement that may control the surface area in flat terrain is the requirement to maintain the clearance between the low point in the gutter and the hydraulic gradient in the storm drain system. For this requirement, the treatment volume and 3-year attenuation volume are constrained on the top by the low point in the gutter minus both the hydraulic gradient clearance and the energy losses in the storm drain system. These volumes are constrained on the bottom by the groundwater elevations (SHWT for this example). The standard hydraulic gradient clearance is given in the Drainage Manual.

The energy losses in the storm drain system can be estimated either of two ways. A hydraulic gradient slope can be assumed. Slopes of 0.05% to 0.1% are common in flat terrain. Multiply the length between the pond and the low point by the assumed slope to obtain the losses. Another approach is to assume a fixed energy loss, ignoring the length between pond and low point. In flat terrain, a reasonable value for this purpose is 0.2 meters.

6. The SHWT elevation is 17.25 meters (18 - 0.75). For this example the energy loss in the storm drain is assumed to be 0.2 m. Then, the treatment and 3-year attenuation volume are constrained to the following height (H).

   H = Low Point in Gutter - Clearance - Estimated Energy Losses - SHWT Elevation
   H = 18.25 - 0.3 - 0.2 - 17.25
   H = 0.5 m

   This is greater than the height (0.45 m) available to "stack" the peak attenuation volume (step 2). Since the 3-year attenuation volume is less than the peak attenuation volume, this constraint will not control the water surface area. If the height was less than determined in step 2, you would estimate the water surface area as done in step 4 except using different values for H and the total volume.

   The water surface area dimensions determined in step 4 apply.

7. Add the maintenance berms to the water surface dimensions. The standard maintenance berm width is given in the Drainage Manual.

Length	$= L_{TOP} + 2$(Berm width)	$= L_{TOP} + 2(6)$	= 64.3 + 12	= 76.3 m
Width	$= W_{TOP} + 2$(Berm width)	$= W_{TOP} + 2(6)$	= 33.3 + 12	= 45.3 m
Area	= 76.3 x 45.3	= 3456 m^2	= 0.346 ha	

8. Increase the value by 10 - 20% to account for preceding information being preliminary. For this example we will increase it 10%.

   Area = 0.346 x 1.1 = 0.38 ha (0.94 ac)

> Realize that this only the pond size estimates. Estimates for access and conveyance must also be made as discussed in the next section.

2.2 Access and Conveyance

The right of way required to convey the project's runoff to and from a pond and to provide access can affect which alternate pond site is the most appropriate. Determine these requirements for each alternate and include the costs and impacts in the evaluation.

Sites placed far from the project will require more right of way to get stormwater to the pond than sites adjacent to the project. Similarly, different pond sites can have different right of way requirements for the outfall (discharge) from the pond. Guidelines for establishing the width or "footprint" of the right of way requirements for conveyance are provided in the Maintenance Section on page 25.

The Department often provides access through the same property obtained for conveying the project's runoff. For pond sites placed far from the project, providing access from a local road closer to the pond is sometimes more reasonable.

The right of way required for access and conveyance is usually obtained as a perpetual easement[3]. Fee Simple right of way may be appropriate sometimes. The opinion of the District Maintenance Office, balanced with property owner preference and right of way costs, is the primary factor for determining which type is appropriate.

2.3 Joint Use (Regional) Facilities

Sometimes the Department and other entities can share a stormwater management facility. Both the Department and the other entities receive the stormwater management benefits of the facility and share in its construction or operation or both. The Department and the other entities enter a written agreement describing the responsibilities of each party. Typically these agreements are made with local governments, but sometimes private entities enter joint use agreements. For example, the Department shares several facilities with golf course owners.

A big advantage of a joint use facility is that the Department can often relieve itself of the maintenance requirements. A joint use facility can have disadvantages such as affecting production schedules, and complicating the permitting and the resolution of non complying discharges, should any occur. When developing a joint use agreement, avoid commitments that hold the Department to completing construction of the site by a certain date because there are often unforseen delays in permitting and funding. Developing an acceptable joint use agreement often requires an extensive coordination effort, involving the project manager and representatives from numerous other offices. Discuss this option with the project manager or District Drainage Engineer.

2.4 Coordination with Property Owners

Often, contacting the property owner to get their preference regarding the shape and location of the pond and location of the access road is beneficial from a right of way standpoint. This coordination is especially important where the Department needs only part of a parcel for a pond. For example, the property owner may prefer a shallower pond although it would require more right of way, or the owner may be interested in reacquiring and maintaining the pond. A certain pond shape could give the owner better use of the remainder of the parcel.

The multi functional team should consider contacting the owner during the evaluation of alternate sites. A situation where contacting the owner during the evaluation may be appropriate is where one person owns all the property in the area. If a contact is not

[3] Refer to Appendix D of the Drainage Manual and the Right of Way Chapter of the Plans Preparation Manual. Both contain additional information about acquisition of property rights.

made during the evaluation process, it is recommended that a contact be made shortly afterward and before starting final design.

Sometimes, contacting the owner may not be appropriate. Where the Department needs an entire parcel, there is no need to obtain preference about location. In this situation, contacting homeowner's associations or abutting property owners may be beneficial to find out if a negative perception of the proposed pond exists.

<u>The project manager with participation from the right of way office should decide, based on individual circumstances, if the owner should be contacted.</u>

The Department's project manager or a right of way specialist or both could make the contact. As drainage designer, you are the best source to answer technical questions and will likely be asked to be present when the contact is made. You cannot provide specifics early in the design process, but you can speak about general principles such as: gentle side slopes and irregular shapes will require more right of way than steep side slopes and a square shape.

When obtained in writing, the property owner's preference should be accommodated to the greatest degree possible. The Department may not be able to accommodate all of the owner's preferences in the design of the pond due to hydraulic constraints or other limitations. However, after weighing and balancing the owner's desires with the other factors, it is likely that some aspect of the owner's preference can be satisfied, thus improving relations during the right of way acquisition process.

If a commitment is made to a property owner, follow through or notify the owner that the Department cannot meet the commitment. Usually, you will not have enough information to commit to anything during the first contact with the owner. Remember that the purpose of the initial contact is to get the owner's preference regarding the shape and location of the pond and location of the access road. The most that you can commit to is to try to accommodate the owner's desires. If, during any discussion, the property owner is told about the operation, shape or location of the pond, this is a commitment. If you subsequently design the pond differently, the property owner should be notified. If the owner is not notified, the right of way specialist is placed in the situation of approaching the owner with a proposed pond configuration that is different from what was previously discussed.

This holds true for changes that occur through the detailed design phase. The owner must be notified if the shape, size, and location of the pond are going to be different from what was previously discussed.

2.5 A Suggested Evaluation Process

An outline for evaluating alternate sites follows, and a flow chart is provided in Figure 2-5. The process is divided into seven main steps of work.

Step 1 Initial Coordination with the Right of Way Office
Step 2 Identify Alternate Drainage Schemes
Step 3 Estimate the Right of Way Required for Each Alternate
Step 4 Team Agrees with the Alternates
Step 5 Estimate Costs and Assess Impacts
Step 6 Summarize Findings
Step 7 Site Selection

The following steps are directed toward the drainage designer, but there is also discussion of activities that other offices should do. Normally, you should do the steps in order; however, it is not intended to be a rigid process. Often, doing certain steps earlier in the process or doing several steps concurrently will be reasonable and prudent. The most important issue is to maintain the coordination necessary to ensure that pond sites are selected using a multi functional team.

The degree of detail will vary with individual projects and between FDOT districts. It is essential that you discuss this with the project manager or the District Drainage Engineer before starting the evaluation.

Step One Initial Coordination with the Right of Way Office

The purpose of this coordination is to provide a preliminary pond size and a general location to the right of way office and to ask the right of way office to identify potential sites.

Shortly after the roadway typical section is set, provide the Right of Way Office with a preliminary estimate of the size and a general location of the pond. Use aerial contour maps, old construction plans, available surveys and other data to identify the primary basins and the general outfall locations (discharge points). Identifying the high points along the project usually separates the primary basins. At this stage, assume that the pond site will be near the lows in the terrain and will be close to the existing outfalls. As a preliminary size estimate, use 20% of the roadway right of way draining to the outfall. The area identified for the general location should be large enough to allow for several alternates to be developed. Refer to Figure 2-1. The project manager should relay this information to the right of way office so they can include the preliminary costs for pond sites in their cost estimates.

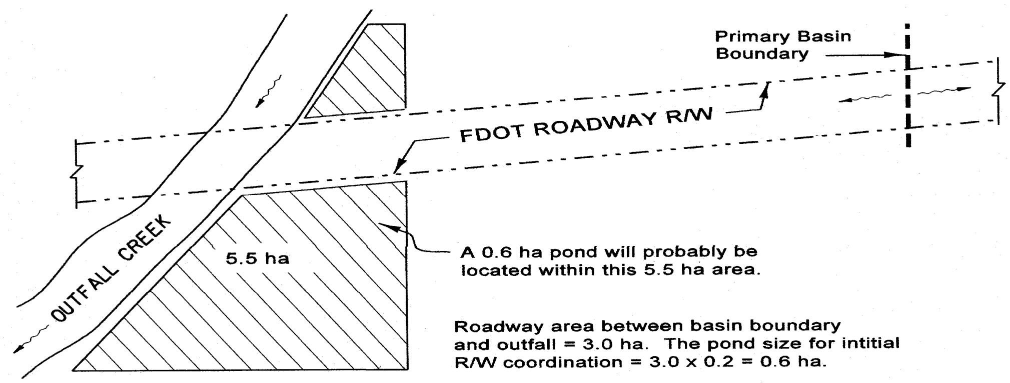


Figure 2-1
Size and Location for Initial
Coordination with the Right of Way Office

When the corridor and alignment (left, right, or center) are set, the project manager should request the right of way office to identify parcels along the roadway[4] that could be economical for a pond, due to the impacts of the roadway footprint. The right of way office should also identify existing excess property in the area.

When the right of way office completes this task, the project manager should arrange a meeting with the team to discuss all potential pond sites[5], aesthetic concerns, and possible contacts with property owners. It is suggested that representatives from right of way, drainage, and environmental management attend.

Step Two Identify Alternate Drainage Schemes

Before developing the alternates, familiarize yourself with soils and groundwater conditions in the area and with the various stormwater quality treatment methods. Use the Natural Resource Conservation Service (NRCS) (formerly Soil Conservation Service) soil surveys to obtain the soil information. The treatment methods are discussed in Chapter 4.

It may be reasonable to start this step by qualitatively eliminating areas that are not hydraulically feasible. For example, such areas may be too high in elevation, or may be at the beginning of the drainage system rather than at the end.

[4] At this stage impacts of the roadway footprint at intersections and interchanges may still be uncertain, simply because the geometry has not been set. These areas may warrant discussions with the right of way office at a later time.

[5] Refer to tax maps while discussing potential pond sites. The project manager should have these, if not, the local government should.

For projects in developing areas, consider contacting the Planning (or Development) Department of the local government to find out the zoning, the planned land use, and if proposed developments exist. Although this information should not automatically eliminate a site from being evaluated, it may help you to identify viable alternates.

Identify two or three alternate drainage schemes for each primary basin. If two or three vacant sites are not available, then consider developed sites. Familiarize yourself with the list of considerations on page 7 when identifying your drainage schemes. Consider the sites identified by the right of way office in Step One. This is not to say that these sites need to be evaluated as alternates. All of the alternates evaluated must be viable. But you should give these sites consideration to be evaluated.

The alternates may be as simple as two different locations for a wet detention pond, or a wet detention pond compared with a dry pond with underdrain at the same location. A system using two ponds, one for off-line quality treatment and one for attenuation could be compared with a single pond designed for both quality and attenuation. In areas with expensive right of way, identifying an alternate that uses a non standard approach such as sand box filters or pumping stations may be prudent. Check with the District Drainage Engineer before doing so. See Figure 2 - 2.

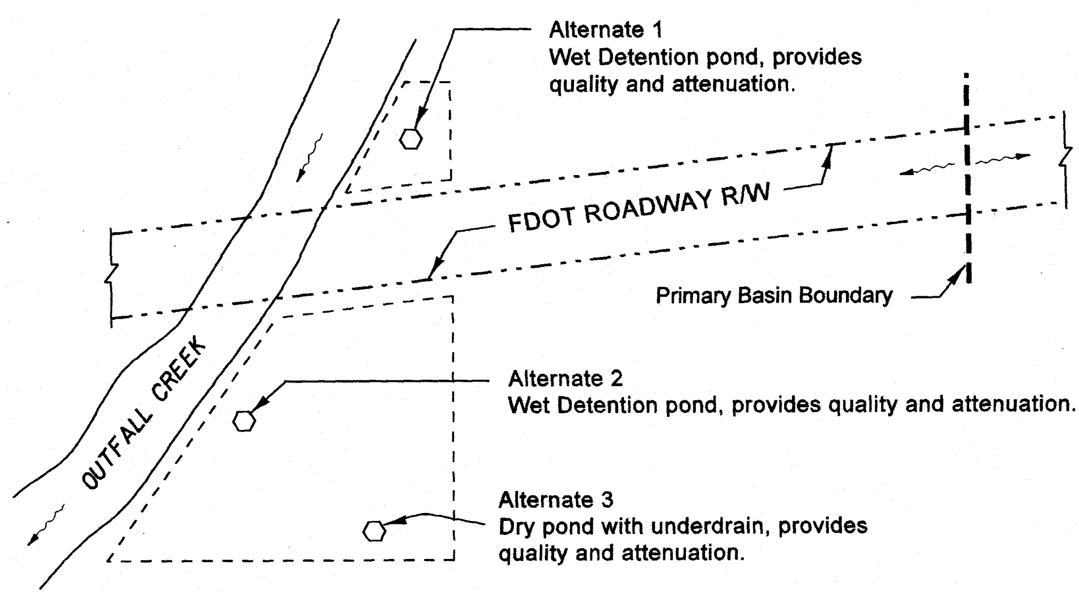


Figure 2-2
Alternate Drainage Schemes

Step Three Estimate the Right of Way Required for Each Alternate

A. Consider the need for additional soils and groundwater information. Most of the Department's districts accept the NRCS soil surveys for pond site evaluations. For alternates in areas of poor soils using retention or exfiltration and for projects discharging to a closed basin, site specific data may be appropriate. If you feel that additional information is warranted, discuss this with the District Drainage Engineer.

Steps B through F apply to ponds discharging to open basins. Ponds discharging to closed basins have the additional complication of assuring that the drawdown requirements are met (See Chapter 5).

B. Determine the required treatment (quality) volume. See the discussion of treatment volumes in Chapter 4. Refer to the appropriate regulatory agencies rules or meet with the agency at this time.

C. Estimate the required attenuation volume. See the discussion of Estimating Attenuation Volume on page 50.

D. Estimate the low point in the proposed roadway. Discuss the grade with the roadway designer as necessary

E. Obtain ground elevations around each alternate site. Using a contour map with one foot intervals is usually sufficient. In flat terrain where one foot contour maps are not available, obtaining a survey of the ground elevations around each alternate site may be appropriate.

F. Determine the pond surface area necessary to satisfy all applicable criteria. Refer to the typical controlling factors on page 8. If you know of aesthetic preferences that will affect the surface area, such as shape, side slopes, landscaping, or preserving existing vegetation, account for them in the surface area determination. Example 2.1 goes through this and the following two steps.

G. Add the maintenance berms to the above area.

H. Increase this area by 10-20% to account for the preceding information being preliminary.

I. Place these surface area requirements within parcel boundaries in a way that minimizes the number of parcels required. For example, avoid using part of two parcels when the pond will fit within one.

J. Determine the right of way requirements for access to the pond and for conveyance to and from the pond.

K. Sketch each alternate site and its requirements for conveyance and access on the tax maps (preferably on aerial background). Refer to Figure 2-3.

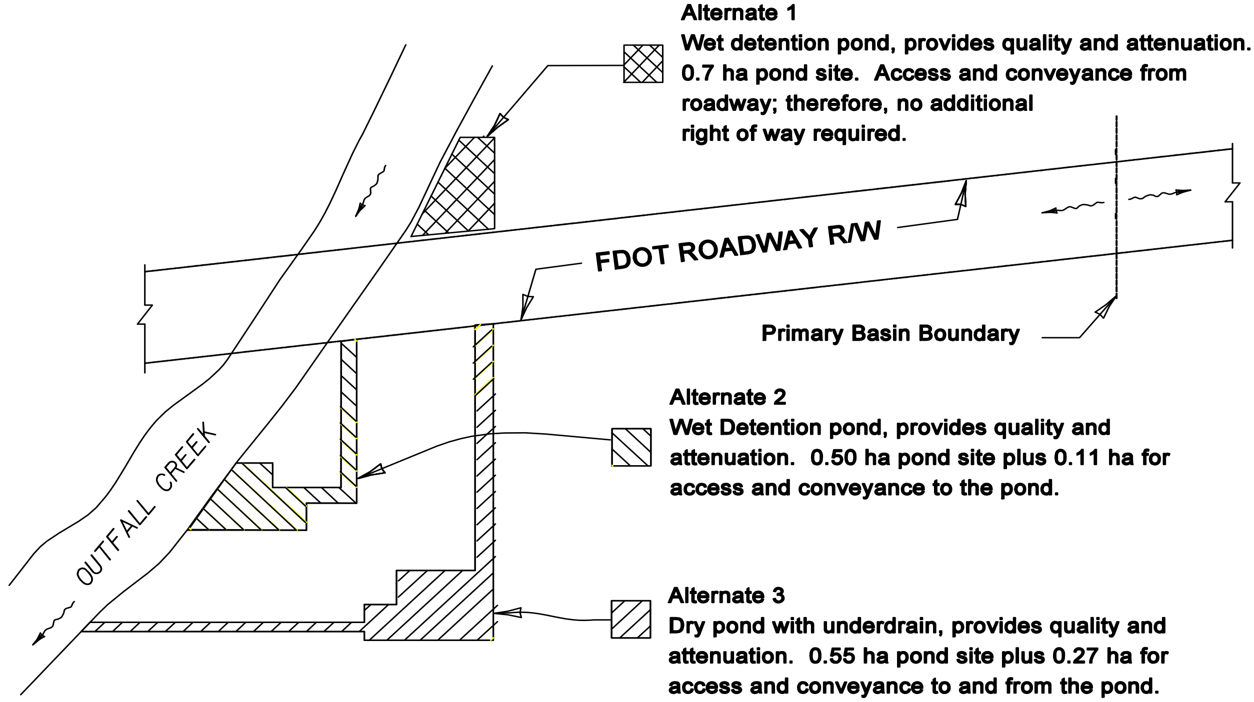


Figure 2-3
Sketch of Each Alternate's Estimated Requirements

Check with the project manager or District Drainage Engineer to see if they want to review the above work before proceeding to the next step.

Step Four Team Agrees with the Alternates

The project manager should arrange a meeting with the team to discuss the alternates. The meeting has several purposes: 1) discuss how the right of way requirements fit within parcel boundaries, 2) confirm that alternates being considered are viable, 3) consider the need to contact property owners to obtain their preference of shape and location, 4) confirm that the access and conveyance requirements are reasonable, and 5) discuss the aesthetic, social, cultural, and environmental impacts of each alternate.

If the property owners are contacted, their preferences should be discussed among the appropriate team members, and the sites appropriately adjusted before proceeding to the next step.

Step Five Estimate Costs and Assess Impacts

Once the team agrees with the alternate drainage schemes, the project manager should request environmental assessments, right of way cost estimates, and utility impact assessments for each alternate site. The purpose of the environmental assessments is to determine potential hazardous material contamination and potential impacts to environmental factors such as threatened, endangered or significant species and cultural resources. Environmental specialists from the Environmental Management Office usually do the assessments, which should include cost estimates associated with any mitigation and environmental clean up.

The purpose of the utility assessment is to determine the existence of utility corridors through each alternate site.

You, as drainage designer, should estimate the construction cost of each alternate including the conveyance requirements to and from the pond. Usually the largest costs are associated with earthwork, pond liner (when required), and pipe. Statewide average unit prices for the standard pay items are provided in the publication “Construction Contract History” which is available for download. (Note: hard copy is not available.) For alternates that are similar, estimating construction cost differences rather than total construction costs may be reasonable. If different alternates are expected to have substantially different maintenance costs, estimate these. Since maintenance costs will be spread over time, it will be necessary to equate these to initial costs using a life cycle analysis. The district and state maintenance offices track unit prices for routine maintenance activities. Contact the District Maintenance Office to obtain the latest prices.

Each alternate should have, at a minimum, cost estimates for right of way. Once the estimates and assessments are complete, the various offices should furnish their findings to you via the project manager.

Step Six Summarize Findings

For each basin, combine the findings of the other offices with your construction cost estimates. Use a summary table similar to Figure 2 - 4 to compare the alternates. The Drainage Manual lists the minimum documentation requirements.

Check with the project manager to see if the district staff wants to review the summary before proceeding to the next step.

Step Seven Site Selection

The team should meet to discuss all alternates and select the most appropriate. Cost, maintainability, constructability, public opinion, aesthetics, and environmental, social, and cultural impacts will affect the selection of a pond site. The team should weigh and balance all factors in their decision. Include documentation of the decision with the summarized findings of the previous step.

2.5.1 Start Final Design

For most projects, the actual right of way requirements will be determined during the final design of the pond. The acquisition of the pond site occurs during the process of acquiring the roadway corridor needs. You should revisit the site evaluation process if the final rights of way requirements are substantially different from those originally estimated. Pond locations frequently change as the final design progresses. Sometimes additional sites are evaluated and occasionally the originally selected site is not used. Any additional evaluations of pond sites should be documented as required by the District Drainage Engineer. All changes in right of way requirements must be coordinated with the right of way office.

Alternate Pond Site Evaluation

Project Decription ______________________ Project Number __________

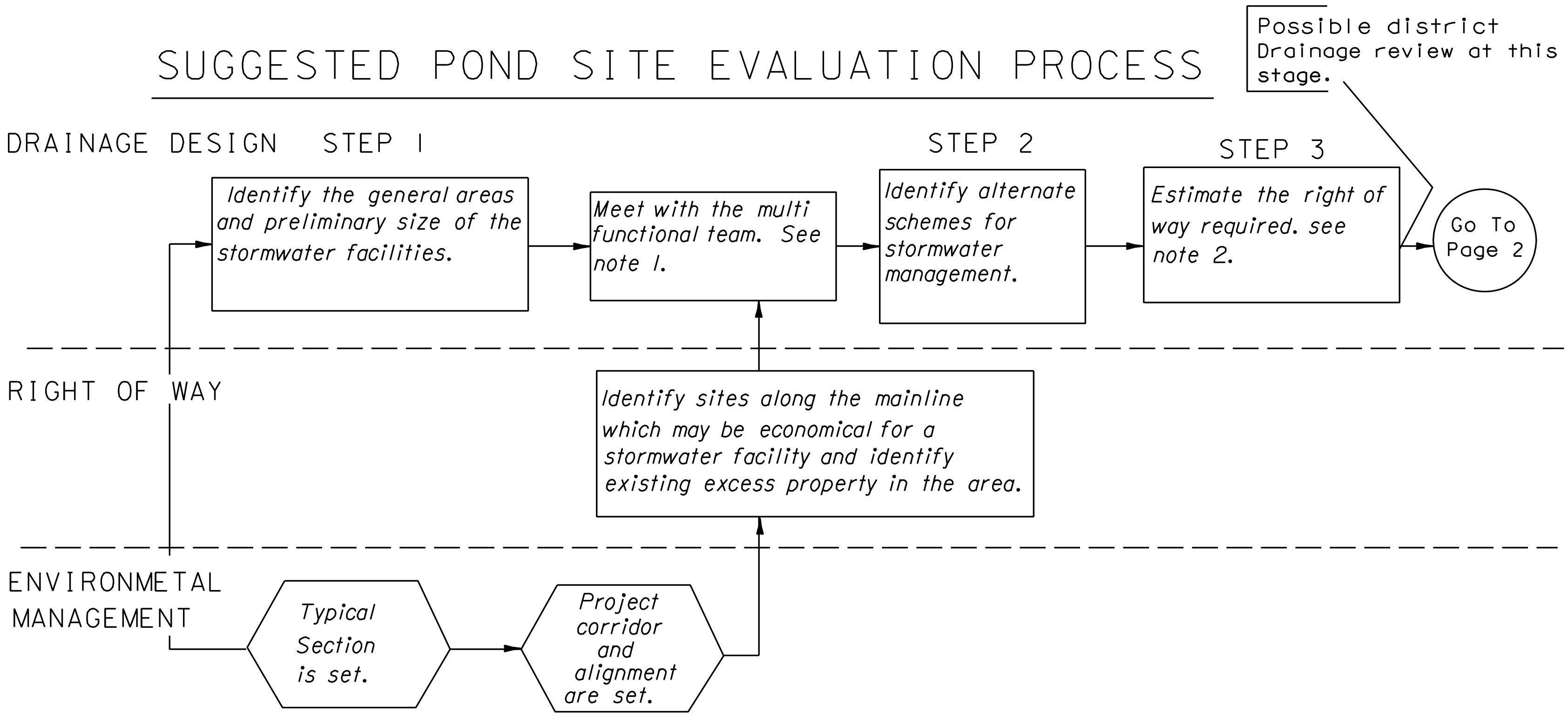
Basin Description ______________________

	Alternate 1		Alternate 2		Alternate 3	
Brief Description of Alternate ►						
	Comments	Costs	Comments	Costs	Comments	Costs
Right of Way						
Construction						
Hazardous Materials						
Utilities						
TES[1] Species						
Maintenance						
Cultural Resources						
Public Opinion						
Aesthetics						
Other						
Total Costs						
Comments, Advantages, Disadvantages, etc.						

1. Threatened, Endangered, or Significant

Figure 2-4

SUGGESTED POND SITE EVALUATION PROCESS



Notes

Note 1. The purpose of this meeting is to discuss all potential pond sites known at this time, asthetic concerns, and possible contacts with property owners. It is suggested that the project manager and representatives from right of way, drainage, and environmental management attend.

Note 2. Input from Design, Landscape Architect, Maintenance, Construction, and other units as necessary.

Figure 2-5, Page 1

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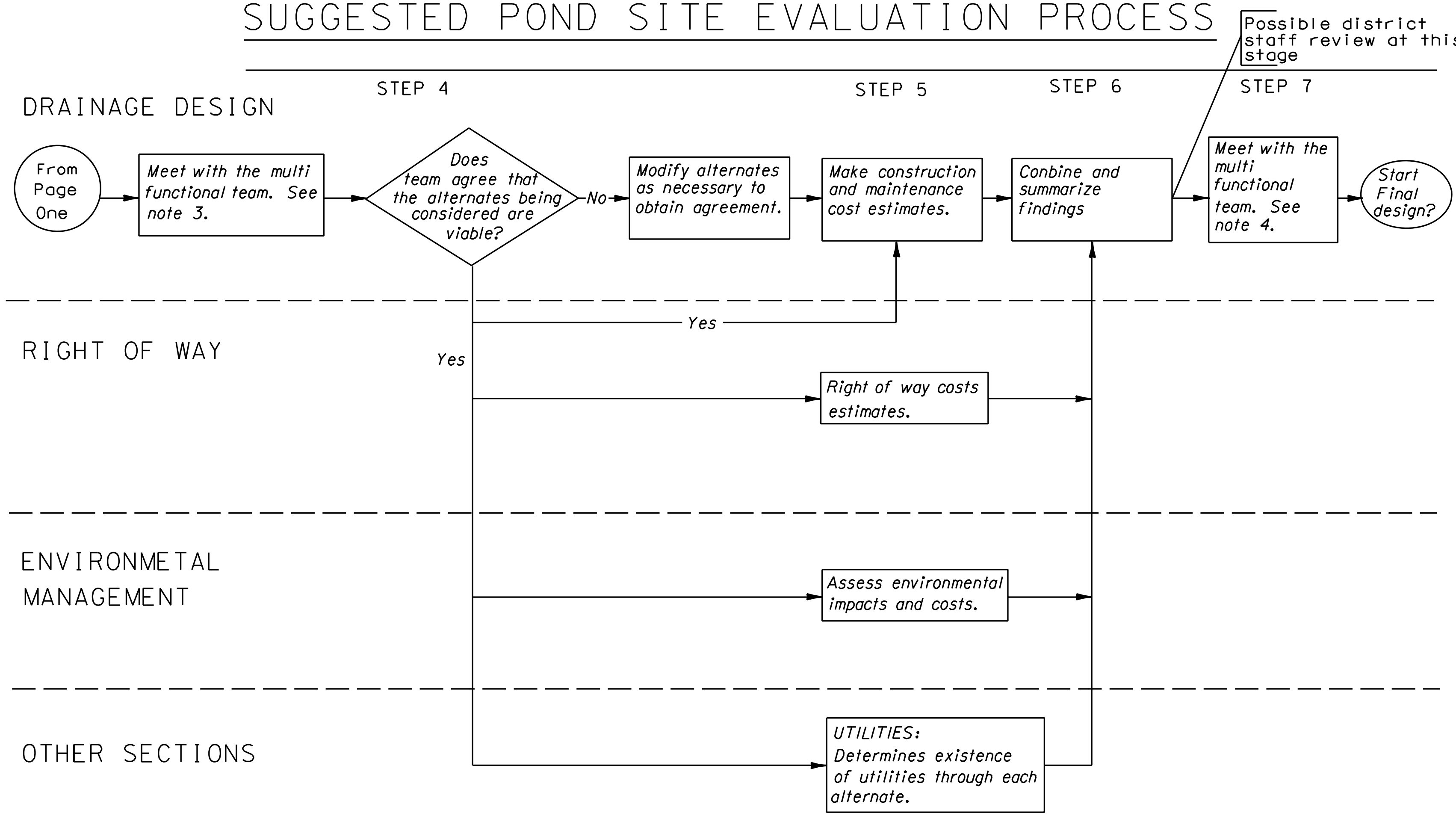
SUGGESTED POND SITE EVALUATION PROCESS



Notes

Note 3. The meeting has several purposes: 1) discuss how right of requirements fit within parcel boundaries, 2) confirm that alternates being considered are viable, 3) consider the need to contact property owners to obtain their preference on shape and location, 4) confirm that the access and conveyance requirements are reasonable, and 5) discuss the aesthetic, social, cultural, and environmental impacts of each alternate.

Note 4. The purpose of this meeting is to discuss the costs and impacts of the alternates and to select an alternate.

Figure 2-5, Page 2
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Chapter 3

Maintenance, Construction, and Aesthetic Concerns

3.1 Maintenance

Maintenance must be a consideration throughout the process of designing a stormwater facility. Long-term maintenance costs are inevitable, but they can be minimized by appropriate consideration during the design of a facility. The difference between a maintainable design and a design that is difficult and expensive to maintain will often be the difference between an attractive operating facility and a neglected, non functioning facility generating frequent public complaints.

3.1.1 Pond Configurations

Side slopes:

> Use a slope of 1(vert) to 4(horiz) or flatter. Steep slopes are harder to mow and are more susceptible to erosion than flat slopes. Slopes steeper than 1:3 must be mowed with special equipment. This is generally more expensive than using regular mowers.

Maintenance berms:

> The Drainage Manual gives the minimum widths and slopes. These are acceptable for most situations.
>
> For ponds that will maintain a permanent or normal pool, keep the lowest point of the maintenance berm at least 0.3 meters (1 ft.) above the top of the treatment volume. This is to minimize saturation of the maintenance berm.

Corners:

> Use a radius of 9 meters (30 feet) or larger for the inside edge of the maintenance berm. This is based on the largest piece of normal maintenance equipment[6]

[6] Several maintence vehicles were modeled using the AUTOTURN program (Transoft Solution, Inc.) The GRADALL 880 required the largest turning radius and gate opening. Special thanks to Don Witmer of District One Maintenance for providing vehicle geometry.

Benchmark:

Have a benchmark (pay item number 580-401) constructed in or near all ponds. It will be used to check critical elevations of the pond and outlet control structure. Avoid installing benchmarks in areas subject to settlement such as high fill sections and areas subject to vehicle loads. An outside corner of the maintenance berm in a minimal fill section would be an appropriate location.

Sediment buildup:

Design the pond with a 1 meter (3 feet) deep sediment sump near the inlet to the pond. In retention ponds (described in Chapter 4) where the groundwater is close to the pond bottom, the depth of the sump may need to be reduced to avoid exposing the groundwater. The area of the sump should be approximately 20% of the pond bottom area.

In retention ponds the sediment is visible, but it often accumulates so slowly that it is difficult to see how much exists. A staff gage (pay item 580-402) placed near the inlet, allows the build up to be measured.

Drawdown of Wet Ponds:

In ponds that will maintain a permanent or normal pool, check to see if the normal elevations of the receiving water body will permit a gravity drawdown. If so, construct a pipe in the bottom of the pond to simplify draining the pond during maintenance operations. The pipe should drain to the outlet control structure or a manhole / junction box which discharges to the receiving water body. See Figure 3 - 1. Include the cost of the pipe, valve, and fittings in the cost of the structure to which the pipe connects.

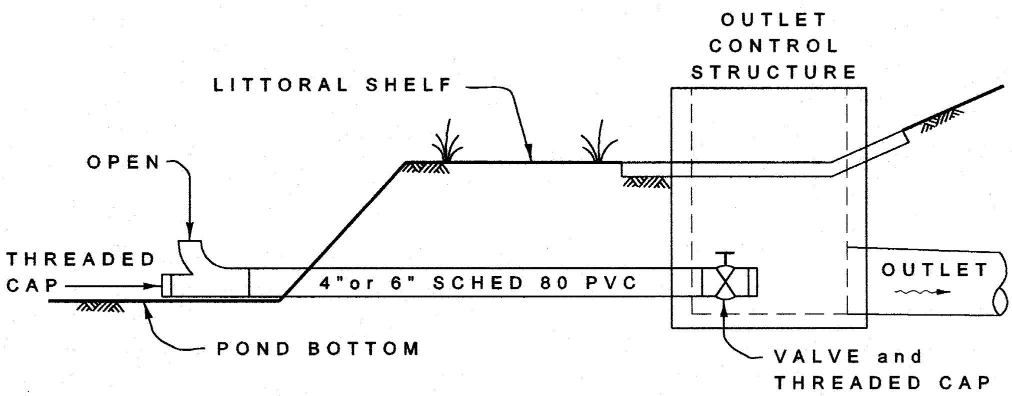


Figure 3-1
Maintenance Drawdown Pipe for Wet Ponds

Permanent (Normal) Pool Depth:

> The main body (not the littoral shelf) of the permanent or normal pool should be deep enough to minimize aquatic growth, but shallow enough to maintain an aerobic environment throughout the water column. The regulatory agencies will usually specify the maximum depth. Typically it is around 2.4 meters (8 feet). If the minimum depth is not specified, use 1.2 meters (4 feet) to minimize aquatic growth.

Side Bank Underdrain Filters:

> Do not construct these around the entire pond. Design the pond to have at least 6 meters (20 feet) of the side slope without underdrain so that maintenance vehicles can get to the pond bottom without running over the underdrain.

3.1.2 Diversion Structures

Diversion structures of off-line systems must have a manhole for access on each side of the weir. Furthermore, the manholes should be located out of the roadway pavement to allow access without blocking traffic. Off-line systems are discussed in Chapter 4.

3.1.3 Conveyance to and from the Pond

The right of way obtained for conveyance to and from the pond must be sufficient to maintain the conveyance. This is true for either piped or open ditch conveyance systems. Figure 3 - 2 provides typical sections for establishing the width of the right of way requirements.

Where the pond discharges to something other than an existing storm drain system, obtain right of way from the pond to a receiving surface water body (lake, wetland, ditch, canal, etc.) even if there are no physical changes proposed to the conveyance path. This assures that the Department will have the right to maintain the flow path.

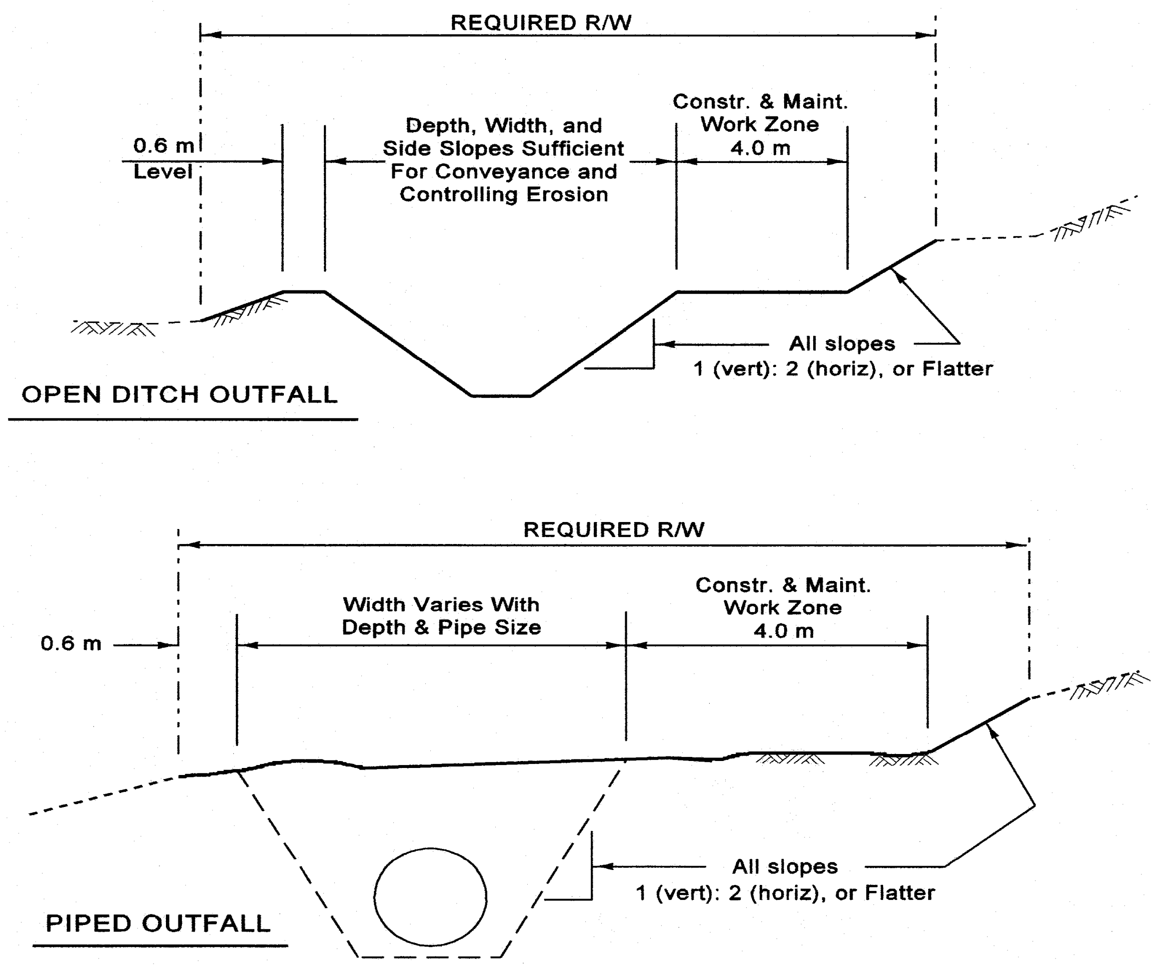


Figure 3-2
Required Right of Way Widths

3.1.4 Vehicle Access

Roads:

Often the right of way used for conveyance will be used to provide maintenance access to the pond. If so, there is obviously no need for additional right of way. For pond sites located far from the project, it may be more reasonable to reach the pond from a local road. In flat terrain, a desirable width of right of way for access only (not including conveyance) is 4.5 meters (15 feet). Larger widths may be necessary for turns. In irregular terrain, consider the distance to tie into natural ground. Concentrated flows crossing the access road may require a culvert crossing. If the vertical clearance is restricted, discuss it with maintenance personnel.

The roadway designer should design and incorporate curb cuts and driveways in the plans where the access road joins the public road.

Gates:

> If the pond is to be fenced, use a 7.3 meter (24 feet) or two 3.6 meter (12 feet) sliding cantilever gates (Index 453). This will allow the largest piece of normal maintenance equipment to enter and exit without having to “back out” the access road. If a swinging gate must be used, the area under the arc of the gate swing should be paved. The gate type, location, and size must be shown in the plans.

3.1.5 NPDES Permits

Active National Pollutant Discharge Elimination System (NPDES) permits may cover the limits of proposed construction. The District NPDES Coordinator needs to review the proposed project to ensure compliance with any active permits.

3.2 Construction

Consider the right of way needed to construct the facility. The right of way needed to maintain the facility, *i.e., the permanent right of way,* may be, but is not always sufficient to construct the facility. If the construction area is outside the permanent right of way, temporary construction access documents should be used to obtain sufficient area for the contractor to construct the facility.

Some water management districts require a professional land surveyor to layout final placement of drainage structures. Some of the Department’s districts are directing the contractor to do this. Discuss this with the project manager or district construction personnel. If they want to have the contractor survey the final placement, include the requirement in the contract documents as directed by the district.

Often the regulatory agencies place special requirements on the Department’s projects as “conditions of the permit.” Such requirements that will affect the contractor’s work must be incorporated into the plans or specifications for bidding and payment purposes. It is not sufficient that the permits will become part of the contract documents.

3.2.1 Structure Tolerances

Unless otherwise dictated, the tolerance for drainage structures is controlled by Section 5-3 of the Standard Specifications. It says “***reasonably close conformity with the lines, grades, . . . specified in the contract documents.”*** The tolerance is particularly important for weirs, orifices and other flow control openings of outlet control structures. Weir dimensions can be calculated quite precisely within the calculations, but it is not reasonable to construct concrete structures to that same precision. Complicating this in the past, the regulatory agencies inspectors have sometimes expected the dimensions to be exactly as shown in the plans.

To address this, specify a tolerance in the plans. A tolerance of plus or minus 0.05 feet is as tight as should be specified. Normally a small tolerance like this does not noticeable effect the performance of the outlet control structure. That is, the discharge rate is not usually sensitive to such small changes to the weir dimensions.

If during your design, you realize that the discharge is sensitive to such small changes in weir dimensions, you should conservatively account for the tolerance in the calculations. For example, to maintain the discharge rate at or below the allowable rate, specify a weir width that is 0.05 feet smaller than the width required to discharge at the allowable rate. And include the tolerance mentioned above. If the weir is constructed 0.05 feet wider than specified, it will match the desired width. If the weir is constructed 0.05 feet narrower than specified, the discharge rate will still be less than the 0.05 feet maximum allowed. In the last condition, you should check that stage has not increased to a point where the pond is now discharging through the overflow point.

Although not often used, another option is to use “bolt on weir plates” with slotted bolt holes. The plate elevation can then be adjusted to exact elevations after the structure is set.

3.2.2 Earthwork Tolerances

By standard specifications, the tolerance for earthwork within a stormwater management facility is 0.3 feet above or below plan cross section (Specification section 120-11). For some retention ponds, having a bottom 0.3 feet higher than anticipated may substantially reduce the treatment volume and somewhat affect the attenuation capacity. Conversely, having a bottom 0.3 feet lower than anticipated may substantially increase the retention (or treated) volume and effect the recovery time. This tolerance will not affect wet-detention facilities.

Do not specify a tolerance that may conflict with the standard specifications. If the standard tolerance will substantially reduce the retention or treatment volume as in a shallow retention pond, design the pond to allow for the bottom being 0.3 feet higher or lower than shown in the plans. In other words, specify a pond bottom that is 0.3 feet lower than necessary to retain the minimum volume. For example, the pond bottom may need to be 0.7 feet below the weir to provide the treatment volume. Specify the bottom to be 1.0 feet below the weir to allow for the earthwork tolerance. Determine the recovery time assuming that the pond bottom is 1.25 feet below the weir, i.e., 0.3 feet below the specified bottom elevation.

This extra effort should be reserved for facilities where the earthwork tolerance could substantially reduce the retention or treatment volume.

3.2.3 Retention System Construction
(Retention Systems are described in Chapter 4.)

Since stormwater management systems are typically constructed during the initial phases of site development, retention basins often receive runoff from the construction site. Stormwater runoff during construction contains considerable amounts of organics, clays, silts, suspended solids, trash, and other material which can clog the bottom surface. During a storm these materials are washed into the retention basin reducing the effectiveness or making the basin inoperable before completion of the project. To address this, add a note similar to the following to the general notes of the plan and profile sheet.

> *Stormwater retention pond xxx [identify the pond(s) as it is (they are) in the plans] is designed to infiltrate stormwater. The pond is susceptible to clogging from construction sediments; therefore, it requires a special excavation procedure as follows. Initially excavate the retention pond to an elevation that is approximately 0.3 meters (1 foot) higher than the final pond bottom elevation. After the drainage area contributing to the pond is stabilized, excavate the interior side slopes and pond bottom to the final elevation. This soil must be removed in a manner to ensure that the silt, clay, organic, and other fine sediments that have washed into the pond during construction are removed. Once the pond has been excavated to final elevation, scarify the entire bottom for optimal infiltration.*

3.2.4 Underdrain Construction

Like retention systems, underdrain systems are very susceptible to construction silts and sediment. To address this, underdrains should be constructed after the drainage area contributing to the pond has been fully stabilized. Since underdrains are usually constructed in soils with low infiltration rates, providing temporary measures to recover the storage volume may be appropriate. Detail these measures in the plans and add a note similar to the following to the general notes of the plan and profile sheet.

> *Stormwater pond xxx [identify the pond(s) as it is (they are) in the plans] contains underdrain systems. These systems are designed to infiltrate stormwater. They are very susceptible to clogging from construction sediments; therefore, they require a special sequence of construction. Initially excavate the pond to rough grade. After the drainage area contributing to the pond is stabilized, construct the underdrains and excavate the pond to the final elevation.*

3.3 Aesthetics

The Florida Legislature has developed legislation directing the Department to include aesthetic considerations into the design aspects of highways. The specific legislation is listed in Chapter 15 of the Project Development & Environmental Manual. Chapter 15 also summarizes the requirements and provides direction in applying them to Department projects. The chapter is directed toward the project development phase, but the same approach should be taken during the final design of a pond. Aesthetic commitments are sometimes made during the project development phase. If so, the environmental document should contain a discussion of visual impacts and aesthetic requirements of the stormwater ponds. Discuss this with the EMO project manager.

The aesthetic quality of a pond is affected by the location, size, shape, side slopes, fencing, and surrounding vegetation. In general, irregular shapes, gradual slopes, and no fence are more aesthetically pleasing and have less visual impact than rectangular shapes and steep slopes with a chain link fence. Irregular side slopes can be used for permanently wet ponds to create an undulating water edge even when the perimeter of the site is rectangular. Preserving existing vegetation[7] and placing native and wetland vegetation can greatly improve the visual acceptance of a pond.

In urban areas, ponds designed with a park like appearance will encourage the local government to undertake the maintenance. If a pond site is to be landscaped, a memorandum of agreement (MOA) must be executed with the local government. An exception to requiring a MOA can be made when the landscape improvement requires no maintenance. The District Landscape Architect / Landscape Manager is familiar with the MOA procedure. Any landscape projects should be coordinated by the project manager with support from the District Landscape Architect / Landscape Manager.

The shape, depth, and side slopes will affect how much right of way is required for a pond. Therefore, aesthetics must be evaluated and weighed among the other factors during the site selection process (Chapter 2). Refer to the District Landscape Architect / Landscape Manager for coordinating and developing appropriate aesthetic features. Your responsibility is to ensure that the design constraints (volumes, depths, littoral shelves) are met while accommodating the aesthetic features. Coordinate with the District Landscape Architect / Landscape Manager to establish the quantity of right of way needed to meet aesthetic and design constraints.

3.3.1 Fence

From an aesthetic point of view, usually the first preference is avoiding the use of fence; the second preference is using special fence to match the community, and the third preference is the standard FDOT fence.

Design stormwater ponds to avoid the need for fence, if feasible. Typically, the flow velocities within a stormwater pond are low and therefore the velocities do not create a hazard. Unexpected deep standing water can be another hazard; however, if you provide flat side slopes, you can minimize this. The FDOT Drainage Manual, FDEP

[7] Preserving existing vegetation usually requires that physical barriers be designed and constructed to protect the area from construction equipment.

Rule 62-25 and all the water management districts' rules allow for unfenced facilities if the slopes are 1(vertical) to 4(horizontal) or flatter. Although the slopes and velocities may not warrant a fence, other conditions may. Refer to Appendix D of the Drainage Manual for further discussion of protective treatment.

When it is necessary to provide a fence, one that fits the surrounding community is often desirable. The style (wood, block, chain link, wrought iron, etc.) will vary from community to community. Pay item 2550-73 exists for special fencing; however, special details and specifications will need to be included in the contract documents. Because of the extra work, special fencing has not been commonly used. Another complication with special fencing is that the Department's maintenance units do not normally have the materials to repair them.

If it is not feasible to provide a special fence, the next option is to use standard FDOT fence. In rural areas, the Type A fence, Index 451, is usually appropriate. In urban areas, Type B fence (chain link), Index 452, is usually appropriate.

Fence Color:

> One of the simplest things that can be done to reduce the visual impact of chain link fence is to specify that it be color coated. Standard Specification Section 966 states that vinyl coated fence fabric be a soft gray color; however, you can specify the color to be dark green or black as allowed by AASHTO M 181. The posts, rails and fittings can also be color coated. To specify color-coated fence, use a pay item footnote (2550-2-xxa thru 2550-6-xxa as applicable) similar to the following.
>
> *The fence fabric, posts, rails, and fittings around the stormwater facilities shall be color coated with xxx (state the desired color) PVC. The PVC coating of the posts and rails shall be in addition to the standard metallic coating and shall meet the requirements of ASTM F 1043. The PVC coating of the fittings shall meet the requirements of ASTM F 626. The cost of the coating is to be included in the cost of these items.*

Fence Height and Barbed Wire Attachments:

> The Department has no requirement for the height of the fence surrounding stormwater facilities. Nor does the Department require the use of barbed wire attachments on fences surrounding stormwater facilities. Other regulatory agencies may have applicable requirements regarding fence height and barbed wire attachments. If so, the Department should comply.

3.3.2 Debris collection

For ponds which are required to be aesthetic, discuss with maintenance personnel and the District Landscape Architect / Landscape Manager the need for collecting debris near the inflow pipe to the pond to prevent the debris from spreading. If it is desirable to collect the debris, direct it to one location where maintenance personnel can easily remove it. Figure 3 - 3 shows some possible configurations.

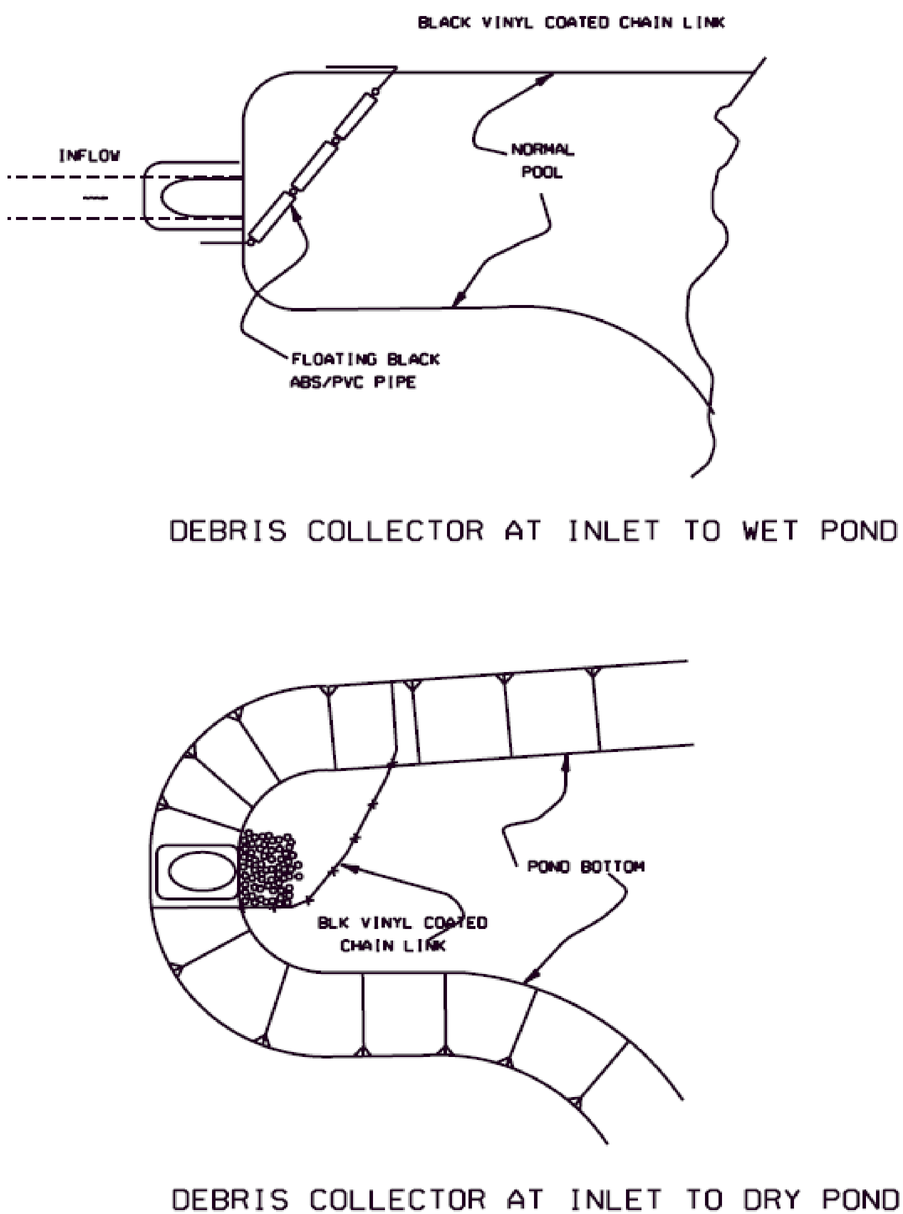


Figure 3-3

Chapter 4

Stormwater Quality

4.1 Design Criteria

The Florida Department of Environmental Protection, the Water Management Districts and the delegated local governments have established design criteria for the operations of stormwater management facilities. There are two main categories of criteria, 1) water quality and 2) water quantity (see Chapter 5). The criteria related to water quality were based on research of rainfall and runoff in Florida and were established to meet state water quality standards. See Appendix A for a discussion of the development of the typical criteria.

Although the criteria are similar around the state, there is some variation. It is essential that you be familiar with the applicable agency's criteria. Read their manuals and coordinate as necessary. Arrange a pre-application meeting to review status of applicable rules and to identify potential problems and concerns to be addressed during design. Agencies usually have checklists and standard forms to be completed for a stormwater permit. Review these forms and address the items relating to stormwater management.

4.1.1 Treatment Volumes

Pollutants in stormwater runoff from urbanized areas generally exhibit a "first flush" effect. This is a phenomenon where the concentrations of pollutants in stormwater runoff are highest during the early part of the storm with concentrations declining as the runoff continues. Substantial reductions in pollutants loads to the state's waters will occur when this first flush is captured and treated. Therefore, each method of treatment requires that a volume of runoff be captured and treated before discharging to surface or ground water. This volume is called the treatment volume.

In general, the treatment volume will vary depending on the classification of the receiving water body and whether the volume is captured on-line or off-line. Sensitive water bodies such as shellfish harvesting waters (Class II) and Outstanding Florida Waters require a larger treatment volume. The classification of the receiving water body should be identified in the Project Development phase as a part of the water quality impact evaluation. FDEP includes a list of sensitive water bodies in Rule 62-302, F.A.C.

4.1.2 Special Conditions

Some of the Department's districts have agreements with regulatory agencies regarding treatment requirements for certain types of highway improvements, such as bridge widening and intersection improvements. Check with the District Drainage Engineer to see if the project is covered by an agreement.

Replacement Treatment may be an option. Sometimes limited or very expensive right of way creates hardship conditions in which it is unrealistic to provide the standard treatment. The Department can sometimes arrange to provide replacement treatment. This means that the Department will treat runoff from an area that presently does not receive any treatment. Providing this treatment compensates for not providing the standard treatment in the area where the hardship condition exists. Treating a larger volume of runoff at another location (drainage area) on the project is usually not considered replacement treatment.

4.2 Concerns of Off-Line Systems

Although off-line treatment systems are preferred from a water quality standpoint and sometimes require less treatment volume, they can complicate the design. Off-line systems are designed to bypass essentially all additional stormwater runoff volumes greater than the treatment volume to the receiving water or an attenuation basin. The bypass flow must pass over the weir of the diversion structure. This can present design problems in that the weir may need to be very long to keep the hydraulic gradient at an acceptable level. And skimmers need to be constructed in front of the weirs, further complicating the practicality of long weirs.

Another concern is that there will be some additional attenuation storage in the off-line basin associated with the hydraulic gradient of the peak flow passing over the weir. When there is significant attenuation storage above the treatment volume, there is a concern that the system will function more as on-line than off-line due to mixing. Metal or rubberized flap gates could be used to address this concern, but they can be a maintenance problem and a noncompliance issue, if not carefully designed.

The outlet control structures of off-line systems are difficult to maintain simply because they are normally placed in junction boxes. They are not seen nor reached as easily as the outlet control structures of on-line systems.

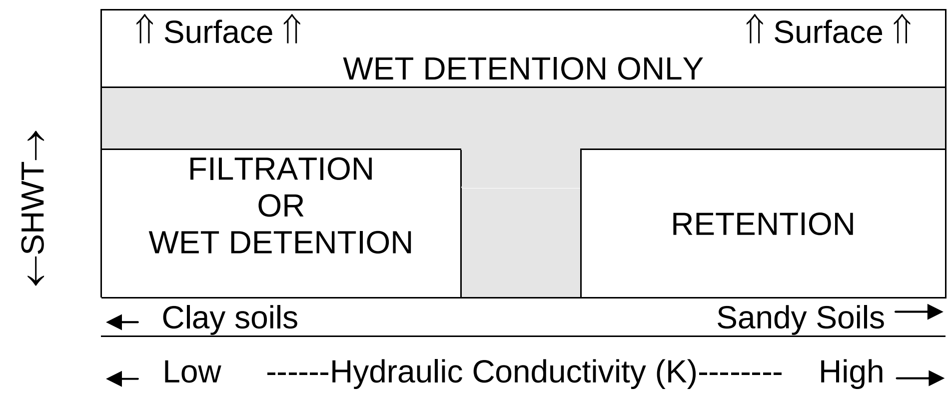
4.3 Seasonal High Water Table

The seasonal high water table elevation (SHWT) is critical to the operation of all of the treatment methods described below. The control (or normal) water elevation of wet detention systems is related to, and sometimes set at, the SHWT. The SHWT is a critical factor in calculating the recovery time of the treatment volume in a retention system. For filtration systems, the lowest point of the underdrain pipe should be at least 0.3 m (1 ft) above the SHWT.

Use the NRCS soil surveys, project specific soil investigations, and field observations (vegetative indicators, observation wells, etc.) to estimate the SHWT.

4.4 Treatment Methods

The treatment methods most commonly used by the Department are wet detention, retention, filtration, and exfiltration. Exfiltration is not discussed in this version of the handbook. The type of soil and the SHWT control the selection of the treatment method. The following figure provides qualitative guidance for the selection.



As shown, wet detention is the only option in areas where the SHWT is near the surface. However, wet detention may also be appropriate in areas where the SHWT is far from the surface and clay soils exist. The use of retention requires that the SHWT be far from the surface and that sandy soils exist. Filtration requires that the SHWT be far from the surface unless impermeable liners[8] are used. Filtration systems can also be used in clay soils. Specific values cannot be applied to this figure because site specific factors such as pond shape, groundwater boundary conditions, and drainage basin characteristics need to be considered. Situations exist where both filtration and wet detention are suitable. In these cases, the Department should weigh and balance other factors such as right of way costs, property owner preference, and long-term maintenance costs to select the most appropriate treatment method.

[8] The Department does not encourage the use of liners, although their use is justified sometimes. Consult the District Drainage Engineer before proposing liners.

4.4.1 Wet Detention Systems

These systems are permanently wet ponds which are designed to slowly release the treatment volume through the outlet control structure. The pollutants are removed by physical, biological, and chemical assimilation. Specifically, pollutant removal processes which occur within the permanent pool include uptake of nutrients by algae and wetland vegetation, adsorption of nutrients and heavy metals onto bottom sediments, biological oxidation of organic materials, and sedimentation.

Advantages	Disadvantages
1. Very effective at removing dissolved and suspended pollutants.	1. Treatment requirements are typically double the requirements for retention and filtration.
2. High probability to function as designed.	2. Depth of the treatment volume is limited to 0.45 meters.
3. Recovery of treatment volume is easily predicted.	3. Because of the above items, rights of way requirements are greater than other methods.
4. Easy and low cost long term maintenance.	4. Often require planting of the littoral zone.
	5. A potential mosquito habitat.

Despite the disadvantages, the Department encourages the use of wet detention. The regulatory agencies usually encourage wet detention systems because they are more effective at treating runoff.

The average length to width ratio of the pond should be at least 2:1. Maximize the flow path of water from the inlet to the outlet to promote good mixing and avoid “dead” storage areas. If short flow paths are unavoidable, use the littoral shelf to increase the effective flow path, provided this is acceptable to the regulatory agency. Figure 4 - 1 shows examples of pond configurations.

The regulatory agency may require the shelf to be planted. If so, consult with the District Landscape Architect / Landscape Manager.

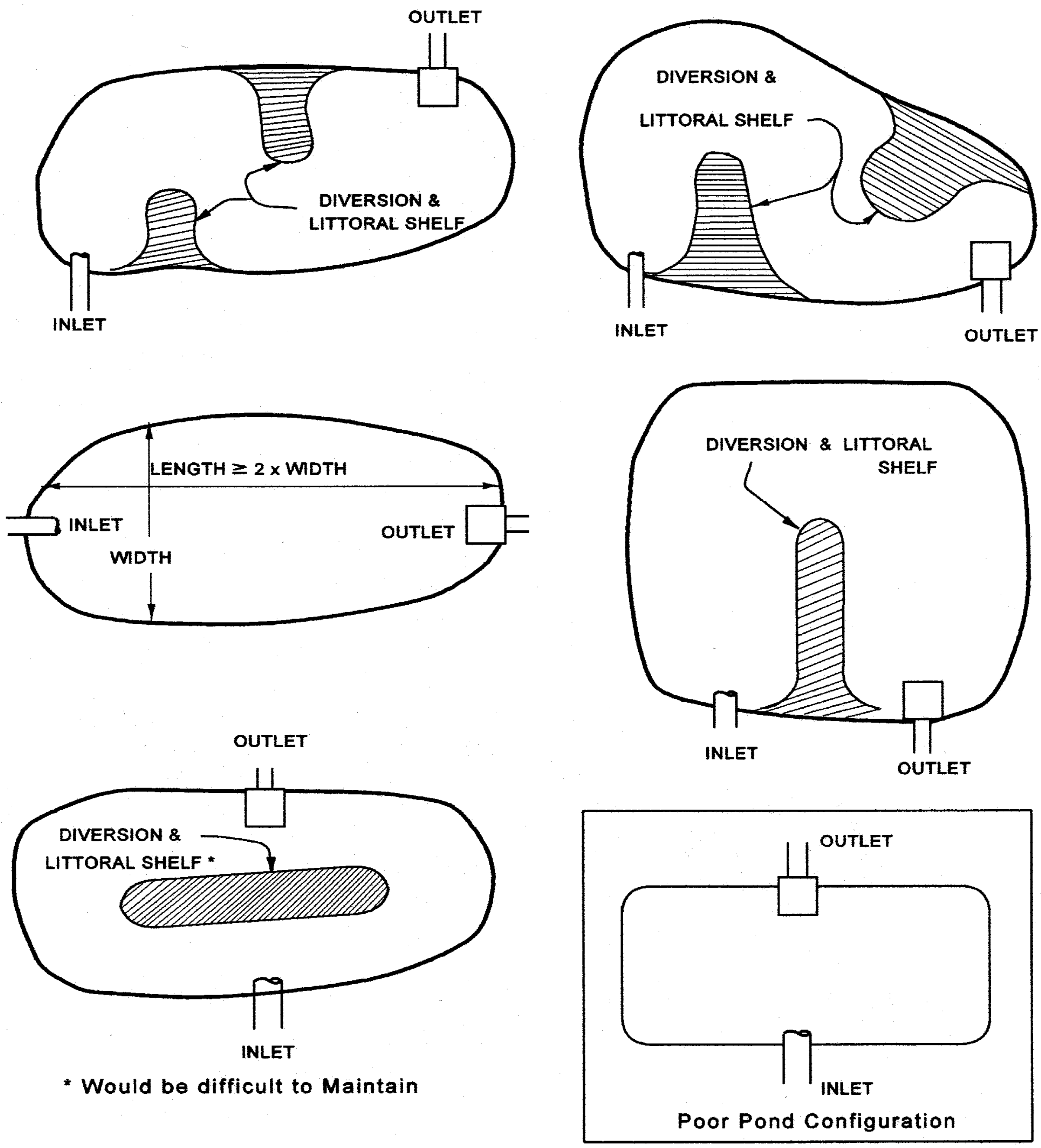


Figure 4-1
Wet Detention Configurations

4.4.2 Retention Systems

A retention system is designed to store the treatment volume, allowing it to infiltrate into the soil. Soil permeability, water table conditions, and the depth to any confining layer must be such that the retention system can infiltrate the treatment volume within a specified time following a storm event. After drawdown has been completed, the basin does not hold water, thus the system is normally "dry." Unlike wet detention systems, and filtration systems, the treatment volume for retention systems is not discharged to surface waters.

Most regulatory agencies require that the treatment volume be available in 72 hours after a storm. See page 65 or a discussion of groundwater flow from retention systems and a recommended approach to modeling recovery of the treatment volume.

4.4.3 Filtration / Underdrain Systems

A filtration system is designed to store the treatment volume, allowing it to pass through a sand filter. It differs from a retention system in that the treatment volume is not infiltrated into the soil but instead discharged to surface water. After passing through the sand filter, the water collects in perforated pipes which discharge to surface water. The Department's standard underdrain is shown in the Roadway and Traffic Design Standard number 286.

Compared with the previous two treatment methods discussed, underdrains are the least reliable. They are subject to clogging during and after construction and are difficult to maintain. Vehicle loads can crush the underdrain pipes. The Department realizes that using underdrains is sometimes necessary, but encourages a thorough evaluation of other treatment methods first.

Configuration:

Where side bank underdrain (Type Va) is used, slope the pond bottom up from the underdrain. This will reduce the saturated soil condition and localized ponding associated with a flat pond bottom. It also increases the chances of sustaining a stand of grass on the bottom. See Figure 4 - 2.

If feasible, construct underdrains out of the primary flow path to avoid directing debris and sediments to them.

To account for construction tolerances, the underdrain pipe should be placed on a slope. Specify flow lines for the pipe at the beginning, at bends, and at the end of the underdrain. In all but very short runs of underdrain, the flow line should drop 150 mm (6 inches) or more to account for construction tolerances.

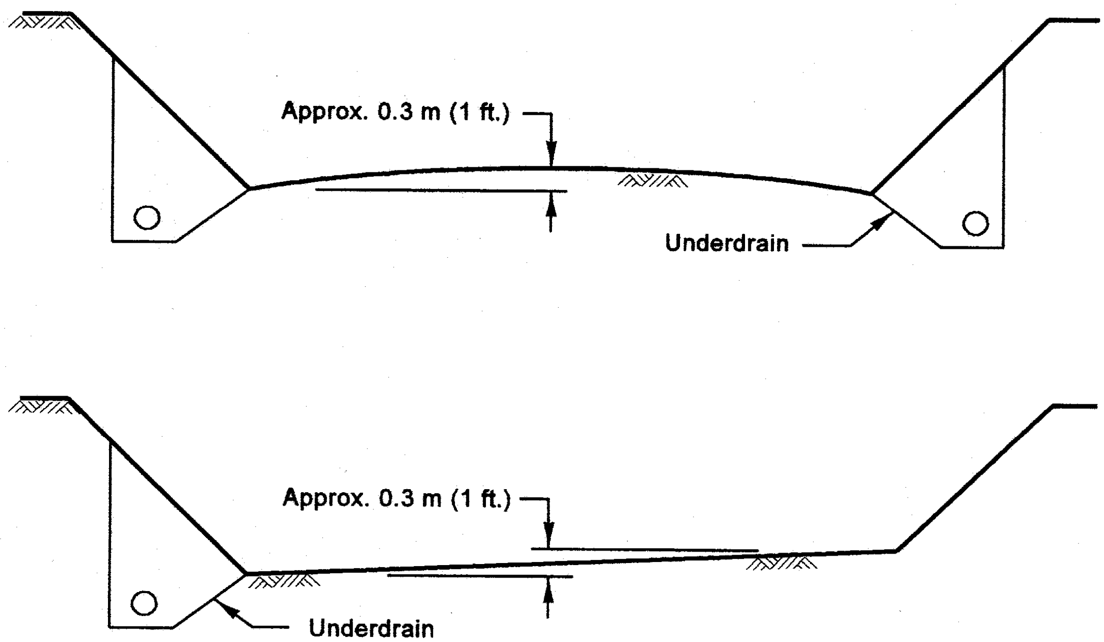


Figure 4-2
Bottom Configurations with Side Bank Underdrains

4.4.3.1 Design Technique

Hydraulic Conductivity of the Fine Aggregate Media:

> For design purposes, use K = 0.004 cm/s (0.5 ft/hr) as the hydraulic conductivity of the fine aggregate media. This does not include the factor of safety of two required by the regulatory agencies. That factor of safety does not have to be applied to the hydraulic conductivity. It is sometimes applied to the length of the underdrain or to the time to drawdown the treatment volume. The above value could be refined by experience from permeability testing of locally available fine aggregate meeting the requirements of the standard specifications for underdrain filter material.

Determining the length of underdrain required is a trial and error process and can be accomplished by the following procedure used with Table 4-2.

1. Develop incremental storage volumes from the maximum elevation of retention storage (i.e., lowest elevation of the outlet control structure) down to the pond bottom. Record these in Columns 1 through 3 of Table 4-2.

2. Determine the effective head (H_E), the average flow length (L_{AVG}), and the average width (W_{AVG}) for flow paths through the underdrain. Determine these for each water surface elevation considered in Step 1. See the discussion following Step 10 for a suggested approach to determining these values. Record these in columns 4 through 6.
3. Calculate the hydraulic gradient (i) for each water surface elevation considered in Step 1 using the values determined in Step 3, and record the results in Column 7. Hydraulic gradient (I) = H_E / L_{AVG} .
4. Assume an underdrain pipe length (L) and calculate the area of filter (A) for each water surface elevation considered in Step 1. Record results in Column 8.
5. Calculate the Darcy flow (Q) using the hydraulic conductivity (K), the hydraulic gradient, and the filter area for for each water surface elevation considered in Step 1. Record results in Column 9.
6. Calculate the average flow rate for each depth interval and record results in Columns 10.
7. Divide the incremental storage volume (ΔV) from Column 3 by the average flow rate from Column 10 to obtain the incremental time (ΔT) to draw down that storage volume. Record results in Column 11.
8. Sum the incremental drawdown times recorded in Column 11 to obtain the drawdown time (ΣT). Record results in Column 12.
9. If the total computed drawdown is longer than required, increase the underdrain length and return to Step 5.
10. Size the underdrain pipe to handle the design flow rate.

Determining the Effective Head, Average Flow Length, and Average Width:

Bottom Underdrain (Type Vb):

For the effective head (H_E) at a given water surface, use the vertical distance from the water surface to the bottom of the fine aggregate material. For the average flow length (L_{AVG}) through the filter, use the depth of fine aggregate, 610 mm (2 feet). For the average width (W_{AVG}) of filter normal to flow use the standard width of 450 mm (1.5 feet) unless non standard geometry is used.

Side Bank Underdrains (Type Va):

The standard index shows the upper and lower limit to side bank underdrain. Try to avoid using the upper limit configuration because of its limited flow capacity in low head conditions. There is very little head and the length of the filter material through which the water must pass is long, resulting in a very small hydraulic gradient.

1. Make a scaled drawing of the average cross section geometry. One is shown in Figure 4-3. The average should represent the midpoint between the high and low end of the underdrain.

2. For the effective head (H_E) at a given water surface elevation, use the vertical distance from water surface to the pipe centerline. At high heads this is non-conservative because the free draining effect of the course aggregate reduces the head. At low heads this is a reasonable assumption.

3. For the average flow length (L_{AVG}) through the filter at a given water surface, use the average of the several straight-line distances from the outside of the pipe to the top of the fine aggregate. This is conservative because it ignores the course aggregate, which is relatively free draining. Refer to Figure 4-3 and Table 4-1 for an example.

4. For the average width (W_{AVG}) of filter normal to flow use the average of the saturated fine aggregate area. Due to the complex transition between vertical and horizontal flow, this is best determined by "visually" estimating the average width based on your scaled drawing. Refer to Figure 4-3 and Table 4-1 for an example.

> The combined effect of using H_E & L_{AVG} as described here should result in conservative flow rates in low head conditions and reasonable rates in high head conditions. At high heads the non conservatism of using the effective head (H_E) to the center line of the pipe is offset by using a average length (L_{AVG}) that is longer than the actual distance through the fine aggregate. At low heads the conservatism of using a longer than actual average length (L_{AVG}) is justified because this zone of the filter is most likely to receive sediment and clog.

Table 4-1
Average Flow Length and Average Width through Side Bank Underdrain

Water Surface Elevation	L_{AVG} Avg. Flow Length Through Filter	W_{AVG} Avg. Width of Filter Normal to Flow
WSE-5 or above	(L5 + L4 + L3 + L2 + L1 + L0) / 6	W to W5
WSE-4	(L4 + L3 + L2 + L1 + L0) / 5	W to W4
WSE-3	(L3 + L2 + L1 + L0) / 4	W to W3
WSE-2	(L2 +L1 + L0) / 3	W2A to W2B
WSE-1	(L1 + L0) / 2	W1A to W1B
Refer to Figure 4-3.		

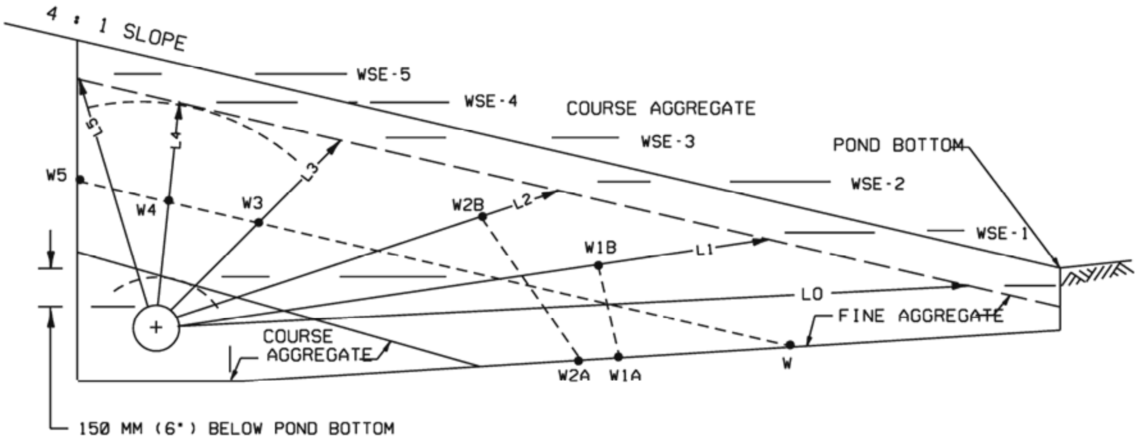


FIGURE 4-3
SIDE BANK UNDERDRAIN (SHOWN 6" BELOW UPPER LIMIT)

DRAWDOWN WORKSHEET FOR UNDERDRAIN

(Using Darcy's Law for Flow-Through Porous Material)

1	2	3	4	5	6	7	8	9	10	11	12
Water Surface Elevation (NGVD)	V Total Volume (ft^3)	ΔV Incr. Volume (ft^3)	H_E Effective Head (ft)	L_{AVG} Avg. Flow Length Through Filter (ft)	W_{AVG} Avg. Width of Filter Normal to Flow (ft)	Hydraulic Gradient $i = H_E / L_{AVG}$ (ft/ft)	Area of Filter $A = L \cdot W_{AVG}$ (ft^2)	Darcy Flow $Q = KiA$ (ft^3/hr)	Average Flow $\frac{Q_1 + Q_2}{2}$ (ft^3/hr)	ΔT Incr. Time (hr)	ΣT Total Time (hr)

Assumed Underdrain Length L = ______________ ft

Hydraulic Conductivity K = ______________ ft/hr
(fine aggregate)

TABLE 4-2

Stormwater Management Facility Handbook 1999

Chapter 5

Stormwater Quantity Control

5.1 The Department's Design Storms

A problem with developing a design storm distribution is that actual storms have an unlimited combination of durations and intensity patterns. What should the duration of the design storm be? Should the peak rainfall occur near the beginning, in the middle, or near the end of the storm? Should there be multiple peaks?

Most of the current widely used rainfall distributions address this by nesting short duration, high intensity storms in the middle of a long duration storm, although very intense peaks do not usually occur in long storms. The largest intensity value is usually placed in the middle of the storm pattern with the remaining values placed alternately before and after this point in order of decreasing intensity. The various NRCS distributions, the SFWMD 3-day and the SJRWMD 4-day distributions are examples of design storm distributions created using this approach. These "nested" distributions are not indicative of actual rainfall patterns and subsequently may produce inaccurate representations of actual runoff characteristics.

These distributions have been used in the past for the design of conveyance systems because they give conservatively high runoff estimates. But, when these distributions are used to determine the pre-developed discharge they can overestimate it. In the developed condition the outlet control structure would be designed to pass the "overestimated pre-developed discharge"; therefore, discharging more in the post developed condition.

Another problem with these distributions is that different drainage areas will react differently to the same rainfall pattern. Small basins with short times of concentration and little storage will have higher runoff rates from short intense storms than from long duration low intensity storms. Long duration, low intensity storms do not usually cause peak discharges from small basins. The opposite is true for large basins. Very large basins with large amounts of storage will have less runoff from short intense storms than from long duration, low intensity storms. Large river systems and static water bodies such as lakes reach peak stages when extreme antecedent conditions exist and variations in intensity usually do not affect their stages.

To overcome the concerns of a single design storm distribution, the SRWMD developed a series of design distributions to better reflect actual rainfall patterns. They developed distributions for 1, 2, 4, 8-hour and 1, 3, 7, 10 day storms using NOAA hourly and sub hourly data. SRWMD requires the use of these distributions for projects within the district.

5.1.1 Rule Chapter 14-86 Florida Administrative Code

In 1986, the Department established rule 14-86 F.A.C. requiring adjacent developments to maintain discharges to at or below pre-developed discharges using a multiple storm approach. In the Department's Drainage Connection Handbook (February 1987), the SRWMD design distributions mentioned above were accepted as appropriate for the entire state. These distributions are included as part of an appendix of the Drainage Manual.

In a July 1988 memorandum, the State Roadway Design Engineer, directed the districts to design the Department's stormwater management systems to rule 14-86. In October 1992, the Drainage Manual was revised. Included in that revision was the requirement to design the Department's stormwater management systems to comply with rule 14-86. The requirement remains in the current Drainage Manual.

5.1.2 Critical Duration

Since the time Rule 14-86 was developed there have been two interpretations of the critical duration and how to apply the multiple storm concept. The definition of critical duration (shown below) as defined in Rule 14-86 lends itself to two interpretations.

> "Critical Duration" means the duration of a specific storm event (i.e., 100 year storm) which creates the largest volume or highest rate of net stormwater runoff (post-development runoff less pre-development runoff) for typical durations up through and including the 10-day duration event. The critical duration is determined by comparing various durations of the specified storm and calculating the peak rate and volume of runoff from each. The duration resulting in the highest peak rate or largest total volume is the "critical duration" storm.

5.1.2.1 Peak Discharge Approach

Duration	Pre-Dev Runoff XX year event	Acceptable Post Dev Runoff XX year event
1 hour	65	
2 hour	70	60
4 hour	66	70
8 hour	60	65
24 hour	30	35
3 day	25	
7 day	24	
10 day	21	

This interpretation of critical duration and the multiple storm concept allows a post developed runoff rate, for a given frequency, that is equal to or less than the highest pre-developed runoff rate of any duration. For example, given the pre-developed runoff rates shown in the table to the right, the allowable runoff rate would be 70, regardless of the duration associated with the peak post developed runoff rate. The post developed runoff rates shown are acceptable because none are greater than 70. You need only run enough durations in the post developed condition to be assured that runoff rates of the other durations do not exceed the allowable.

This approach is consistent with the last sentence of the definition of critical duration. *"The duration resulting in the highest peak rate . . . is the critical duration."* With this approach the pre-developed critical duration can be different from the post developed critical duration, as shown in the values above. Also the pre-developed runoff rate could be calculated with the rational method (Q = CIA) for small basins; therefore, it would not be directly associated with any of the eight durations. The examples in the Drainage Connection Handbook follow this interpretation.

The above discussion is directed toward discharges to open basins. For discharges to closed basins, a similar approach is used with an additional constraint on the runoff volume. For a given frequency, the allowable post developed runoff volume is the largest pre-developed runoff volume of any duration. When using the NRCS technique for computing runoff, the 10-day duration event will always produce the largest runoff volume and therefore be the critical duration. But, for other more refined approaches to modeling infiltration, the critical duration could be something other than the 10-day duration.

5.1.2.2 Storm for Storm Approach (Preferred)

Duration	Pre-Dev Runoff XX year event	Acceptable Post Dev Runoff XX year event
1 hour	65	60
2 hour	70	68
4 hour	66	66
8 hour	60	57
24 hour	30	26
3 day	25	23
7 day	24	22
10 day	21	20

This interpretation of critical duration and the multiple storm concept requires, for a given frequency, that the post developed runoff rate for each duration be less than or equal to the pre-developed runoff rate of corresponding duration. For example, in the table to the right, the allowable runoff rate for each duration is the pre-developed runoff rate. The post developed runoff rates shown are acceptable because they are all less than or equal to pre-developed runoff rate of corresponding duration. The 4-hour duration is critical because it most closely matches the pre-developed runoff rate.

This approach is consistent with the first sentence of the definition of critical duration. *“Critical Duration means the duration . . . which creates the . . . highest rate of net stormwater runoff (post development runoff less pre-development runoff) . . ..”* In the example above, when you subtract the pre-development runoff rate from the corresponding post development runoff rate, all the “net stormwater runoff” values are negative except the 4-hour duration, which has zero “net stormwater runoff.” So the 4-hour duration has the highest rate of net stormwater runoff; therefore, it is the critical duration. This approach is better than the peak discharge approach where the release timing of the facility is critical. FHWA’s Hydraulic Engineering Circular No. 22 (HEC-22) contains a discussion of the concern for release timing.

The above discussion is directed toward discharges to open basins. For discharges to closed basins a similar approach is used with an additional constraint on the runoff volume. For a given frequency, the post developed runoff volumes for each duration can not exceed the pre-developed runoff volumes of corresponding duration.

Although both the “Peak Discharge” and the “Storm for Storm” approaches have been applied to FDOT projects in the past. The Department prefers that you use “Storm for Storm” approach on its projects. The examples in this handbook are based on the “Storm for Storm” approach.

5.1.3 Storm Frequencies

The previous sections primarily discuss durations and the multiple storm concept. Rule 14-86 [14-86.003 (3)(c) 2 & 3] requires that we consider various rainfall event frequencies up to and including the 100 year. The rule does not say that all frequencies must be evaluated.

The more frequent FDOT design storms (2 year - 50 year) do not usually control the size of the pond because the runoff from these storms is less than the 100 year storm. The purpose of evaluating the less frequent storms is to ensure that the pre-developed discharges are not exceeded. And so it becomes a check of the operation of the outlet control structure under various rainfall event frequencies.

Where the discharge is controlled by a simple rectangular weir (one with a constant width), it may be reasonable to run only the 2, 25, and 100 year events. Where the discharge is controlled by a complex weir (width varies with elevation), an orifice, a pipe, tailwater conditions, or any combination of these, evaluate all frequencies (2, 5, 10, 25, 50, 100). Some programs can run all the frequencies at once. If such programs are available to you, run all the frequencies, regardless of oulet control structure configuration.

5.2 Estimating Attenuation Volume

A first step in estimating attenuation volume is identifying outfalls and their associated drainage basin. At this stage consider if it will be necessary to divert runoff from one basin to another. Although the Department does not encourage diverting runoff, doing so sometimes allows the Department to provide stormwater management (treatment and attenuation) in more economical locations. For example, an economical parcel for a pond site may be available in one drainage basin while the parcels in an adjacent basin are very expensive. Diverting some roadway runoff to the “economical parcel” basin from the “expensive parcel” basin may be more economical even when other costs such as construction and maintenance are considered. Before you propose diverting runoff, be sure it is acceptable to the regulatory agency.

When diverting runoff, be careful how you calculate the allowable discharge. Base your allowable (pre-developed) discharge calculations on the pre-developed drainage area that discharges to the proposed outfall. If an area does not drain to the proposed outfall in the pre-developed condition, do not include that area in the allowable (pre-developed) discharge calculations. Therefore, in a basin you divert runoff to the pre-developed drainage area is smaller than the post developed drainage area. Conversely, in a basin you divert runoff <u>from</u>, the pre-developed drainage area is larger than the post developed drainage area.

The actual attenuation volume can not be determined until you “route” the design storms and design the pond. There are several methods for estimating the attenuation volume. The methods more commonly used on the Department’s projects are discussed below.

5.2.1 Pre Versus Post Runoff Volume

A common technique for estimating attenuation volume is to calculate the difference in runoff volume between the post developed and the pre-developed conditions using the NRCS equation for runoff.

$$Q_R = \frac{(P - 0.2S)^2}{P + 0.8S}$$

As written, this assumes the initial abstraction (I_a) = 0.2S & S =(1000/CN) - 10

where: Q_R= runoff depth (in)
P= rainfall depth (in). Use the 100 year - 24 hour depth for evaluating alternate drainage schemes or pond sites.
S = maximum retention or soil storage (in)
CN= watershed curve number

The runoff volume is determined from: VOL = $Q_R \cdot$ Drainage Area

A similar approach can be taken using the Rational Method.

$$VOL = (C_{POST} - C_{PRE}) \cdot P \cdot \text{Drainage Area}$$

An advantage of this technique is that it does not involve any design storm distributions. So there is no concern for which storm duration is critical. On the other hand this technique ignores the timing differences between the pre-developed and post developed hydrographs. As a result, it may underestimate the attenuation volume.

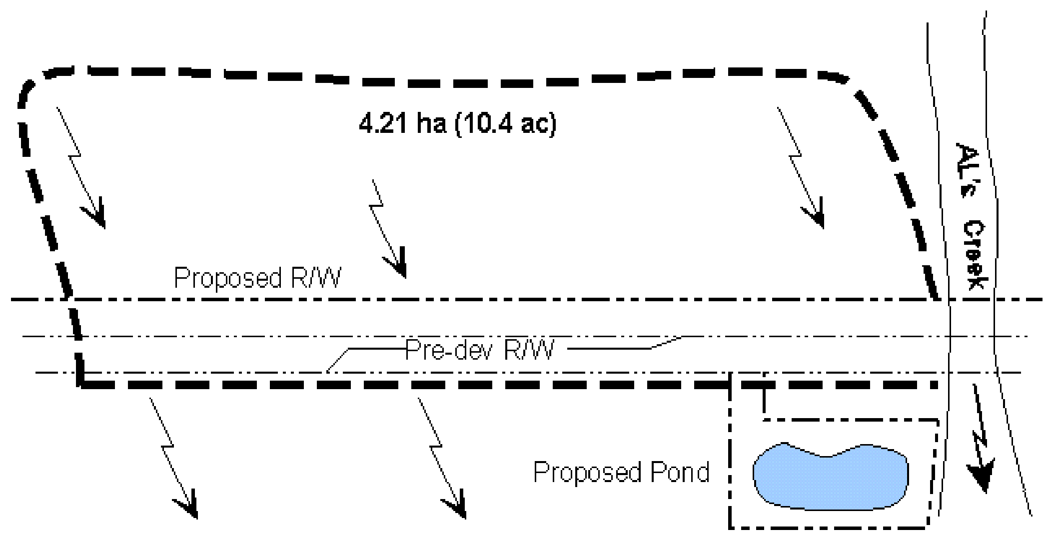
Example 5.1 Estimating Attenuation Volume Using Differences in Runoff Volume

Given:

- Pre-developed Rdwy Pvmt = 2-10 foot lanes
- Drainage Area: Includes Rdwy R/W & Offsite Drainage to Rdwy = 4.21 ha (10.4 ac)

For preliminary pond sizing, use the information from the old drainage map unless you have reason not to.

- Offsite land use = Residential lots averaging 0.20 ha (1/2 ac)
- Proposed Typical Section = 5-lane urban section; Combined Rdwy, Curb, & SW Width = 25.2m (83 ft.)
- Proposed R/W width = 30 m (100 ft.)
- Length of Rdwy within Drainage Area= 520 m (1706 ft.)
- Offsite runoff draining to the project will be taken through the pond, not bypassed around.
- Project located in Narcoossee, Florida, Flat terrain <1%, Hydrologic Soil Group B/D, Project drains to open basin.



Example 5.1

Find: The estimated attenuation volume.

1. Pre-developed Area & Curve Number:
   Roadway Pvmt: = 0.32 ha (0.79 ac) @ CN = 98 (6.1m x 520m)
   Pervious = 3.89 ha (9.61 ac) @ CN = 85 (4.21 ha - 0.32 ha)
   Proposed Pond Area = 0.31 ha (0.77 ac) @ CN = 85
   Total = 4.52 ha (11.2 ac) @ CN = 85.9

   Assume the pond area is 20% of Rdwy R/W (0.2 x 520m x 30m = 0.31 ha). For this example, the proposed pond is located outside the area draining to the roadway, thus the pond must be added to the other areas.

   For this example, the roadway right of way to be acquired is within the area draining to the roadway. For your project the acquired R/W may be outside the area draining to the road, thus requiring that the additional R/W be added to the other areas.

2. Post-developed Area and Curve Number:
   Rdwy, Curb, & SW: = 1.31 ha (3.24 ac) @ CN = 98 (25.2m x 520m)
   Pervious = 2.90 ha (7.17 ac) @ CN = 85 (4.21 ha – 1.31 ha)
   Pond = 0.31 ha (0.77 ac) @ CN = 98
   Total = 4.52 ha (11.2 ac) @ CN = 89.7

3. Calculate the difference in runoff volume between the pre and post conditions for the 100 year - 24 hour storm using the NRCS equation for runoff.

   From the precipitation data of Appendix B of the Drainage Manual, the 100-year - 24 hour volume for Narcoossee is 10.7 inches.

   $$Q = (P - 0.2S)^2 \div (P + 0.8S) \qquad \text{where: } S = (1000 \div CN) - 10$$

	Pre	Post
Potential Abstraction (S) =	1.64	1.15
Runoff Depth (Q) in.	8.95	9.44
Runoff Volume (ac-ft) =	8.36	8.81

   Volume Difference = 554 m^3 (0.45 ac-ft).

   The estimated attenuation volume is this volume difference of 554 m^3 (0.45 ac-ft).

5.2.2 Simple Pond Model Procedure

Another technique for estimating attenuation volume is to route a design storm through a simple pond model. It works best with a routing program that allows a rating curve for the stage - discharge relationship and a stage - storage (not area) relationship for the pond configuration. The model should be set up as follows:

- Arbitrarily select pond bottom and top elevations.
- Use two points for the stage - discharge relationship:
  Zero discharge @ Pond bottom, and Allowable discharge @ Pond top.
- Use two points for the stage - storage relationship:
  Zero storage @ Pond bottom, and Estimated storage @ Pond top

As with any routing this is an iterative process. During each iteration the estimated storage volume is changed to bring the routed peak stage close to the top of the pond. The storage volume that causes the peak pond stage to match the top of the pond is the estimated attenuation storage.

This approach is useful when the discharge rate is limited to something other than the pre-developed rate. It is complicated when working with the Department's multiple design storms. Which design storm do you route? The following is suggested to simplify working with the multiple design storms.

- Determine the pre-developed discharges for the 100 year - 1 hour thru 8 hour design storms. Use the smallest of these as the allowable discharge rate. For the storm for storm approach to critical duration, the post developed discharge rate will be limited to all of the corresponding pre-developed rates, so using the rate for estimating purposes is reasonable. The basis for running only the 1-8 hour is that one of these design storms is usually critical to sizing ponds discharging to open basins.

- Route the post developed conditions using a "nested" design storm such as the NRCS Type 2 Florida modified or the applicable WMD design storm. These distributions are often as severe or more severe than the Department's distributions.

Example 5.2 Estimating Attenuation volume using a Simple Pond Model

Given:

- The same conditions as in Example 5.1
- Pre developed time of concentration = 29 min.
- Post developed time of concentration= 21 min.

Find: The estimated attenuation volume

1. Pre-developed runoff:

   Determine the pre-developed discharge rates for the 100 year FDOT 1-hour and 8-hour design storms. Using a typical program which uses the NRCS unit hydrograph approach you should obtain values similar to these when using a peak shape factor of 256. The rainfall volumes are tabulated in Step 1 of Example 5.3.

Pre-developed Peak Runoff Rates m^3/s (cfs)			
1 hour - 100 yr	2 hour - 100 yr	4 - hour - 100 yr	8 -hour - 100 yr
0.94 (33.2)	0.85 (30.1)	0.72 (25.5)	0.79 (27.8)

   The discharge associated with the 4-hour storm is the smallest and will be used as the allowable discharge.

2. Develop a simplified pond model as follows.

	Elevation	Discharge m^3/s (cfs)	Storage
Pond Bottom	0	0	0
Top of Pond	10	0.72 (25.5)	Trial and Error

3. Route a nested design storm through the pond using post developed conditions. For this example, we will route the 25 year, SFWMD 72-hour storm. Adjust the storage as necessary to have the routed peak stage match the top of pond. After numerous iterations, a storage value of 1600 m^3 (1.3 ac-ft) was found acceptable, so:

   The estimated attenuation volume is 1600 m^3 (1.3 ac-ft).

5.2.3 Other Techniques

FHWA's Hydraulic Engineering Circular No. 22 (HEC-22) provides several methods for estimating attenuation volume. Examples are provided and comparisons made. Although most of these techniques are reasonably accurate, they, like the previous method, are complicated when working with the Department's multiple design storms.

5.3 Tailwater Conditions

Tailwater conditions can affect the design of the outfall structure, the size of the pond, and even the evaluation of alternate pond sites. The pond must meet the attenuation requirements during the tailwater conditions expected to occur coincident with the design storms. Predicting the tailwater condition can be difficult sometimes. The points to which our facilities discharge are usually associated with watersheds much larger than the drainage area of our facility. It may be appropriate to model the larger watershed and apply design storms to both the road project and the larger watershed simultaneously. This way any timing related effects should be addressed. A simpler approach is to estimate the worse case tailwater condition and see if it submerges the control point of the outlet control structure. If it does not, the tailwater condition can be ignored in the design of the weir / orifice of the outlet control structure.

Placing a pond in a 100-year riverine floodplain can complicate the design due to high tailwater conditions that may be coincident with the design storm. Other complications such as flood plain compensation and changes to floodway conveyances may exist as well. The Department's Drainage Handbook: Bridge Hydraulics addresses impacts to floodway conveyances.

5.4 Routing Calculations

The storage indication method is used by most programs to route hydrographs through stormwater management facilities. FHWA's Hydraulic Engineering Circular No. 22 (HEC-22) contains a discussion of the storage indication method with an example. The Drainage Connection Handbook also discusses this method.

Most engineers currently use computer programs to route hydrographs through stormwater facilities. Although the computer reduces the effort, it does not eliminate the iterative process of modifying the pond and outlet control structure after each run. Numerous iterations are usually required to design an acceptable pond and outlet control structure. There are five items, which can be adjusted to meet the discharge requirements. These are 1) weir width (or orifice size), 2) weir crest (or orifice invert) elevation, 3) pond surface area, 4) pond depth and 5) pond length to width ratio. Although some of these may be constrained by regulatory requirements, the following provides general guidance for making adjustment during the iterative process.

If the only change made is:	The results are:
Increasing weir width (or orifice size)	Increases discharge and lowers stage.
Lowering weir crest (or orifice invert) (1)	Increases discharge (volume more than rate) and lowers peak stage.
Increasing pond surface area (increases storage above and below weir crest)	Decrease discharge and lowers peak stage. For retentions systems, increases infiltration and shortens recovery time.
Lowering pond depth (1) (increases storage below the weir only)	Decreases discharge and lowers peak stage. For retention systems, decreases infiltration and lengthens recovery time when saturated groundwater flow conditions exist.
Increasing length to width ratio	Increases discharge and raises peak stage, due to slight reduction in storage area for the same surface area. For retention systems, increases infiltration and shortens recovery time when saturated groundwater flow conditions exist.
1) Normally applicable to only retention systems.	

5.5 Discharges to Watersheds with Positive Outlet (Open Basins)

Using the storm for storm approach, the Department's criterion for discharges to open basins is that, for a given frequency, the post developed discharge rate for each duration must be less than or equal to the pre-developed discharge rate of corresponding duration. Most of the regulatory agencies also have requirements for post developed discharge rates. These and the Department's criterion must be met.

Example 5.3 Discharge to Watershed with Positive Outlet (Open Basin)

This example uses information developed in Example 2.1, 5.1, and 5.2.

Given: The following information has been verified since the time of the pond site evaluation.

- SHWT Elevation at pond site: = 17.1 meters (56.1 ft) Agreed to by regulatory agency.
- Lowest ground elev. around pond site = 18.0 m (59.1 ft) From design survey.

Find: The required pond configuration to meet the FDOT criterion. For this example, the pond will also be designed to meet SWFWMD, and SFWMD criteria.

1. Determine the rainfall volumes using the IDF curves (for durations less than 1 day) and the precipitation data of Appendix B of the Drainage Manual.

Rainfall Volumes: Narcoossee FL						
	2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
1 hr	2.4	2.95	3.25	3.75	4.1	4.5
2 hr	2.8	3.5	3.9	4.5	5.0	5.5
4 hr	3.3	4.0	4.6	5.4	6.0	6.6
8 hr	3.8	4.9	5.6	6.5	7.3	8.0
1 day	4.8	6.3	7.7	8.7	9.7	10.7
3 day	6.1	7.9	9.1	10.8	12.2	14.1
7 day	7.5	9.4	11.5	13	14.8	16.8
10 day	8.5	11	13	15	17	19

First Round of Iterations

2. Determine the Pre-developed runoff rates: This will establish the allowable discharge rates.

Time of Concentration = 29 min (from Ex. 5.2)
Pre-developed CN:
Roadway Pvmt: = 0.32 ha (0.79 ac) @ CN = 98 (from Ex. 5.1)
Pervious = 3.89 ha (9.61 ac) @ CN = 85(from Ex. 5.1)
Proposed Pond Area = 0.38 ha (0.94 ac) @ CN = 85
Total = 4.59 ha (11.3 ac) @ CN = 85.9

The proposed pond size is from Example 2.1 “pond siting stage.” This is a reasonable assumption for the first iteration.

To simplify this problem, we have used the time of concentration, roadway pavement area, and offsite land use from other examples in this chapter. Actually, the latest information from the design surveys and field reviews of the proposed project should be used to establish the pre-developed conditions. Using a typical program, which uses the NRCS unit hydrograph approach you should obtain values similar to these when using peak shape factor of 256. This peak shape factor is used throughout this example.

For the first round of iterations for a pond discharging to an open basin, it is usually sufficient to run the 100-year FDOT 1-8 hour duration storms and the regulatory agency design storm.

Pre-developed Runoff m^3/s (cfs)					
DOT1 - 100yr	DOT2 - 100yr	DOT4 - 100yr	DOT8 - 100yr	FLT2M - 25yr	SF72 - 25yr
0.95 (33.6)	0.86 (30.4)	0.73 (25.8)	0.80 (28.1)	0.87 (30.6)	1.03 (36.3)

3. Post-developed Runoff:

   In urban sections, the time of concentration is best determined from the storm sewer design tabulations. For this example, assume the storm sewer tabs have a Tc = 21 min.

   Time of Concentration: = 21 min
   Post-developed Area & Curve Number:
   Rdwy, Curb, & SW: = 1.31 ha (3.24 ac) @ CN = 98 (from Ex. 5.1)
   Pervious: = 2.90 ha (7.17 ac) @ CN = 85 (from Ex. 5.1)
   Pond: = 0.38 ha (0.94 ac) @ CN = 98
   Total = 4.59 ha (11.3 ac) @ CN = 89.7

4. Develop a stage - storage relation (pond configuration) for the first round of iterations.

   Dimensions at Peak Stage = 64.3 m by 33.3 m (from Ex. 2.1)

> For the first iteration use the configuration estimated in the pond sitting evaluation unless you have reasons not to.

Peak Stage = 17.7 m (58.1 ft) To maintain freeboard between ground line of 18.0 m (59 ft.)

Although some WMD's allow treatment below SHWT, this example assumes that treatment is above SHWT. Then, the pond length and width at SHWT elevation (for routing purposes the SHWT elevation is considered pond bottom) are:

Bot. Length = Top length - 2 [side slope (peak stage - $elev_{SHWT}$)]
= 64.3 m - 2 [5 (17.7m - 17.1m)] (1:5 side slopes)
= 58.3 m (191 ft)

Similarly, Bot. Width= 27.3 m (90 ft)

Using these dimensions and side slopes, develop a stage - storage relationship. The values to the right were obtained using the equation for the volume of a frustum of a pyramid.

Stage m (ft)	Storage m^3 (ac-ft)
17.10 (56.10)	0 (0.00)
17.22 (56.50)	201 (0.16)
17.34 (56.90)	415 (0.34)
17.46 (57.30)	643 (0.52)
17.59 (57.70)	884 (0.72)
17.71 (58.10)	1140 (0.92)

5. Develop an outfall structure for the first round of iterations. Do so using the maximum allowable stage and discharge. For this example the maximum allowable stage is the ground elevation minus the freeboard [18.0 m – 0.3 m = 17.7 m (58.1 feet)]. The maximum allowable discharge is largest pre-developed discharge; which for this example is the SFWMD 72 hour –25-year design storm (see step 2).

   Weir crest elevation = 17.28 m (56.7 ft) The treatment volume (310 m^3, given in Ex 2.1) stacks 0.18 m high.

   Weir Width (L) $= Q \div (C \times H^{1.5})$ from $Q = C \times L \times H^{1.5}$.
   $= 36.3\text{cfs} \div (3.1 \times 1.37^{1.5})$ The max head = 17.7 m - 17.28 m = 0.42 m (1.37 ft).
   = 2.22 m (7.3 ft.)

   For this example, we have assumed no tailwater effects. For your projects, you will need to consider the effects of the tailwater conditions on the outfall control structure.

   During this round of iterations ignore the effects of the water quality bleed down orifice and start the routings at the top of the treatment volume.

6. Route the selected design storms. Using a typical routing program you should obtain values similar to the following.

	Design Storm		Discharge m^3/s (cfs)	Peak Pond Stage m(ft)
Table 5.3-1 Pond Configuration: Pond Dimensions at SHWT = 58.3 m (191 ft) x 27.3 m (90 ft) SHWT El. = 17.1 meters (56.1 ft) Avg Side Slope = 1: 5 Weir Crest El. = 17.28 m (56.7 ft) Weir Width = 2.22 m (7.3 ft) Starting WS = 17.28 m (56.7 ft) Allowable Stage = 17.71m (58.1ft)	FDOT1hr - 100 year	Pre	0.95 (33.6)	
		Post	1.08 (38.2)	17.71 (58.1)
	FDOT2hr - 100 year	Pre	0.86 (30.4)	
		Post	0.99 (35.0)	17.71 (58.1)
	FDOT4hr - 100 year	Pre	0.73 (25.8)	
		Post	0.82 (28.8)	17.65 (57.9)
	FDOT8hr - 100 year	Pre	0.80 (28.1)	
		Post	0.87 (30.6)	17.65 (57.9)
	SCS-T2FLM - 25 year	Pre	0.87 (30.6)	
		Post	0.93 (33.0)	17.68 (58.0)
	SFWMD-72hr - 25 year	Pre	1.03 (36.3)	
		Post	1.08 (38.0)	17.71 (58.1)

From this table it appears that 100-year, one or two hour may be critical because they exceed the pre-developed discharge more than the others. Overall the configuration used in the first iteration is close to meeting the requirements. The weir length needs to be shortened to decrease the peak discharge. Doing so will cause the stage of the 1-hour, the 2-hour, and the SFWMD design storm to exceed the allowable stage so the pond size needs to be increased also.

After making several runs, the stage - storage relationship shown to the right and a weir width of 1.83 m (6.0 ft) is close to meeting the requirements of the design storms modeled. The second row in the table is the weir crest elevation sufficient to store the treatment volume.

Stage m (ft)	Storage m^3 (ac-ft)
17.10 (56.10)	0 (0.00)
17.19 (56.40)	327 (0.27)
17.22 (56.50)	439 (0.36)
17.34 (56.90)	896 (0.73)
17.46 (57.30)	1374 (1.11)
17.59 (57.70)	1872 (1.52)
17.71 (58.10)	2390 (1.94)

Using this configuration you should obtain values as shown below.

Table 5.3-2 Pond Configuration: Pond Dimensions at SHWT = 88.0 m (288.7 ft) x 40.0 m (131.2 ft) SHWT El. = 17.1 m (56.1 ft) Avg Side Slope = 1: 5 Weir crest El. = 17.19 m (56.40 ft) Weir Width = 1.83 m (6.0 ft) Starting WS = 17.19 m (56.4 ft) Allowable Stage 17.71 (58.1)	Design Storm		Discharge m^3/s (cfs)	Peak Pond Stage m (ft)
	FDOT1hr - 100 year	Pre Post	0.95 (33.6) 0.77 (27.1)	17.59 (57.7)
	FDOT2hr - 100 year	Pre Post	0.86 (30.4) 0.77 (27.3)	17.59 (57.7)
	FDOT4hr - 100 year	Pre Post	0.73 (25.8) 0.75 (26.4)	17.59 (57.7)
	FDOT8hr - 100 year	Pre Post	0.80 (28.1) 0.76 (27.4)	17.59 (57.7)
	SCS-T2FLM - 25 year	Pre Post	0.87 (30.6) 0.78 (27.5)	17.59 (57.7)
	SFWMD-72hr - 25 year	Pre Post	1.03 (36.3) 0.87 (30.7)	17.62 (57.8)

From this table it appears that the 4-hour is critical since it is the only duration that the post developed discharge is not less than the pre-developed discharge. The SFWMD design storm creates the highest stage of the storms modeled.

Second Round of Iterations

7. Adjust the drainage basin characteristics due to the pond size being increased in the previous step. Remember that for this example the pond is outside the area draining to the pond so increasing the pond size also increases the total area. See Example 5.1. During the first iteration, we assumed the entire pond area had a CN = 98. A more refined estimate of the pond area curve number can be made at this time.

Pond Area:
Water Surf Dims at Peak Stage = 94.1m x 46.1m
Water Surface Area at Peak Stage = 0.43 ha (1.06 ac) @ CN = 98
Total pond area (incl maint berms = 0.62 ha (1.53 ac)
Grassed area within total pond area = 0.19 ha (0.47 ac) @ CN = 85

Total Project Area and Curve Number:
Pre-developed CN:
Roadway Pvmt: = 0.32 ha (0.79 ac) @ CN = 98 (same as Step 2)
Pervious: = 3.89 ha (9.61 ac) @ CN = 85 (same as Step 2)
Proposed Pond Area = 0.62 ha (1.53 ac) @ CN = 85
Total = 4.83 ha (11.9 ac) @ CN = 85.9

Post-developed CN:
Rdwy, Curb, & SW: = 1.31 ha (3.24 ac) @ CN = 98 (same as Step 3)
Pervious: = 3.09 ha (7.64 ac) @ CN = 85 [2.9 ha (Step 3) +0.19 ha]
Pond: = 0.43 ha (1.06 ac) @ CN =100
Total = 4.83 ha (11.9 ac) @ CN = 89.9

8. Calculate the pre-developed runoff and then route the design storms. For this example we will add the FDOT 24-hour 100-year design storm at this time. The results are shown in the following table.

Table 5.3-3	Design Storm		Discharge m^3/s (cfs)	Peak Pond Stage m (ft)
Pond Configuration: (Same as previous table) Pond Dimensions at SHWT = 88.0 m (288.7 ft) x 40.0 m (131.2 ft) SHWT El. = 17.71m (56.1ft) Avg Side Slope = 1: 5 Weir Crest El. = 17.19 m (56.40 ft) Weir Width = 1.83 m (6.0 ft) Starting WS = 17.19 m (56.4 ft) Allowable Stage = 17.71 m (58.1ft)	FDOT1hr - 100 year	Pre Post	1.00 (35.2) 0.81 (28.7)	17.61 (57.8)
	FDOT2hr - 100 year	Pre Post	0.90 (31.9) 0.82 (28.9)	17.61 (57.8)
	FDOT4hr - 100 year	Pre Post	0.76 (27.0) 0.79 (27.9)	17.60 (57.7)
	FDOT8hr - 100 year	Pre Post	0.84 (29.5) 0.82 (28.9)	17.61 (57.8)
	FDOT24hr - 100 year	Pre Post	0.32 (11.2) 0.31 (11.1)	17.43 (57.2)
	SCS-T2FLM - 25 year	Pre Post	0.91 (32.1) 0.82 (29.0)	17.61 (57.8)
	SFWMD-72hr - 25 year	Pre Post	1.08 (38.1) 0.92 (32.5)	17.64 (57.9)

From this table we can see the discharge for the 4-hour needs to be reduced and the stage of the SFWMD storm can still be increased, so the weir width can be reduced. After several iterations, a weir 1.37 m (4.5 ft) wide works. The results are as follows.

Table 5.3-4 Pond Configuration: Pond Dimensions at SHWT = 88.0 m (288.7 ft) x 40.0 m (131.2 ft) SHWT El. = 17.71m (56.1ft) Avg Side Slope = 1 : 5 Weir Crest El. = 17.19 m (56.40 ft) Weir Width = 1.37 m (4.5 ft) Starting WS = 17.19 m (56.4 ft) Allowable Stage 17.71 (58.1)	Design Storm		Discharge m^3/s (cfs)	Peak Pond Stage m (ft)
	FDOT1hr - 100 year	Pre Post	1.00 (35.2) 0.73 (25.8)	17.65 (57.9)
	FDOT2hr - 100 year	Pre Post	0.90 (31.9) 0.76 (26.7)	17.65 (57.9)
	FDOT4hr - 100 year	Pre Post	0.76 (27.0) 0.76 (26.8)	17.65 (57.9)
	FDOT8hr - 100 year	Pre Post	0.84 (29.5) 0.78 (27.6)	17.68 (58.0)
	FDOT24hr - 100 year	Pre Post	0.32 (11.2) 0.31 (10.9)	17.46 (57.3)
	SCS-T2FLM - 25 year	Pre Post	0.91 (32.1) 0.78 (27.5)	17.68 (58.0)
	SFWMD-72hr - 25 year	Pre Post	1.08 (38.1) 0.86 (30.3)	17.68 (58.0)

Since this configuration meets the requirements for these design storms, the pond size is probably adequate. We need to make sure that the discharges are not exceeded for the less frequent (2 thru 50-year) DOT design storms. We will also check the longer duration storms though it appears that the long duration storms (24 - 240 hour) will not control the size of the pond, since the stages and discharges of the 24-hour are much less than 1-hour through 8-hour duration storms.

9. Check size of orifice bleed down. For this example a 38 mm (1.5 in) diameter orifice or less is required to meet the typical wet detention criteria [discharge no more than ½ of the treatment volume in 60 hours and discharge the total treatment volume in no less than 120 hours]. At maximum pond stage the discharge through this orifice is less than 0.003 m^3/s (0.1 cfs). This is insignificant for this problem. The orifice flow will be ignored and the routing calculations will be started at the weir crest as done in previous iterations.

> If the discharge through the bleed down orifice at peak stage is small, ignore it. If not, model the orifice in the routing. If the orifice is modeled, the starting water surface should reflect some amount of drawdown. The average inter-event period between storms is 72 hours. Most wet detention systems hold at least ½ of the treatment volume for 60 hours. Therefore for most wet ponds, starting the water surface at an elevation associated with ½ of the treatment volume would be reasonable. If the regulatory requirements allow for a quicker drawdown, it may be reasonable to start the water surface at the bleed down orifice.

10. Run the other design storms. The other design storms were routed through the above pond configuration and all the post developed rates were less than the pre-developed rates, except one. A summary of these is shown below.

TABLE 5.3-5 (Example 5.3)							
Pond Config. as in Table 5.3-4		100 - year Discharge m^3/s (cfs)	50 - year Discharge m^3/s (cfs)	25 - year Discharge m^3/s (cfs)	10 - year Discharge m^3/s (cfs)	5 - year Discharge m^3/s (cfs)	2 - year Discharge m^3/s (cfs)
1-hour	Pre	1.00 (35.2)	0.88 (31.0)	0.78 (27.4)	0.63 (22.3)	0.55 (19.3)	0.40 (14.0)
	Post	0.73 (25.8)	0.63 (22.3)	0.55 (19.4)	0.43 (15.3)	0.37 (13.0)	0.26 (9.1)
2-hour	Pre	0.90 (31.9)	0.79 (28.0)	0.69 (24.2)	0.56 (19.8)	0.48 (16.9)	0.34 (11.9)
	Post	0.76 (26.7)	0.66 (23.3)	0.57 (20.0)	0.46 (16.1)	0.39 (13.7)	0.27 (9.6)
4-hour	Pre	0.76 (27.0)	0.68 (24.0)	0.60 (21.1)	0.48 (17.1)	0.40 (14.2)	0.31 (10.8)
	Post	0.76 (26.8)	0.67 (23.7)	0.59 (20.7)	0.47 (16.8)	0.39 (13.8)	0.30 (10.5)
8 -hour	Pre	0.84 (29.5)	0.74 (26.5)	0.65 (23.0)	0.54 (19.0)	0.45 (16.0)	0.32 (11.3)
	Post	0.78 (27.6)	0.70 (24.5)	0.60 (21.1)	0.49 (17.3)	0.41 (14.3)	0.28 (9.9)
24 –hour	Pre	0.32 (11.2)	0.28 (10.0)	0.25 (8.9)	0.22 (7.7)	0.17 (6.0)	0.12 (4.3)
	Post	0.31 (10.9)	0.28 (9.7)	0.24 (8.6)	0.21 (7.4)	0.17 (5.8)	0.12 (4.1)
3-day	Pre	0.23 (8.2)	0.20 (7.1)	0.18 (6.2)	0.15 (5.2)	0.13 (4.5)	0.10 (3.4)
	Post	0.23 (8.2)	0.20 (7.1)	0.18 (6.2)	0.15 (5.2)	0.13 (4.4)	0.09 (3.3)
7 day	Pre	0.17 (5.9)	0.15 (5.2)	0.13 (4.5)	0.11 (4.0)	0.09 (3.2)	0.07 (2.5)
	Post	0.17 (5.9)	0.15 (5.2)	0.13 (4.5)	0.11 (4.0)	0.09 (3.2)	0.07 (2.6)
10 day	Pre	0.22 (7.8)	0.20 (6.9)	0.17 (6.1)	0.15 (5.3)	0.12 (4.4)	0.09 (3.4)
	Post	0.22 (7.8)	0.20 (6.9)	0.17 (6.1)	0.15 (5.3)	0.13 (4.4)	0.10 (3.4)

The 7-day 2-year post- developed discharge rate is grater than the pre-developed rate. If carried to three significant digits, the increase in 0.02 cfs (2.56-2.54). This is within the accuracy of these calculations and would be acceptable for most projects. If you or your project reviewer are concerned about an increase like this, the weir configuration could be modified slightly as is done in step 8 of Example 5-4.

11. Fine tune pond dimensions.

The stage-storage values used in this example have been based on length and width dimensions applied to a frustum of a pyramid. When you apply the radii to the corners, the storage would be reduced using the same pond dimensions, so use an equivalent stage-area relationship when working with the contours within the Microstation file. Doing so will also allow you to configure the pond for aesthetic purposes while maintaining the necessary stage / storage relationship.

5.6 Discharges to Watersheds without Positive Outlet (Closed Basins)

Using the “Storm for Storm” approach, the Department’s criteria for projects discharging to a closed basin is that, for a given frequency, the post developed discharge (rate and volume) for each duration must be less than or equal to the pre-developed discharge (rate and volume) of corresponding duration.

The retention volume shall be large enough to ensure that the post developed discharge volumes do not exceed the pre-developed discharge volumes. The retention volume is the volume between the pond bottom and lowest discharge elevation of outlet control structure.

When using the NRCS runoff methodology, the retention volume can be conservatively calculated as the difference between the pre-developed and post developed discharge volume for the 100-year 10-day event. Some of this volume is infiltrated into the soil during the storm so, the actual retention volume can be and is sometimes less than this. During long duration design storms such as the 3 through10 day, the volume infiltrated during the storm can be substantial. It is recommended that you account for this by using a program that models the infiltration while routing the storm hydrograph. When doing so, the required retention volume is not known until you have routed the storms and know how much volume infiltrates during the storm event.

The retention volume must recover at a rate such that ½ of the volume is available in 7 days and the total volume available in 30 days. When measuring the volume recovered, the pond is instantly (or over a very short time) filled with a runoff volume equal to the retention volume. The water is then allowed to infiltrate with no inflow to the pond.

5.6.1 Retention System Groundwater Flow Analysis

The following approach is based on the current approach to modeling the recovery of the treatment volume in retention systems. The same techniques are applicable to the infiltration of retention systems discharging to closed basins. The next several pages summarize the critical information contained in the following documents. We suggest that you read these documents before designing a retention system.

a) *"Stormwater Retention Pond Infiltration Analyses in Unconfined Aquifers".* Prepared by Jammal and Associates, Inc 1989, for the SWFWMD, Brooksville, FL. At the time of printing this handbook, the document was available from Andreyev Engineering, Inc., Sanford FL, (407) 330-7763.

b) "*Full-Scale Hydrologic Monitoring of Stormwater Retention Ponds and Recommended Hydro-Geotechnical Design Methodologies*" Prepared by PSI, Jammal and Associates Division, for the SJRWMD, August 1993, Special Publication SJ93-SP10. At the time of printing this handbook, the report was available from the SJRWMD library in Palatka.

During a storm event, runoff from the drainage basin enters the pond and infiltrates the pond bottom. At the beginning of a storm, the groundwater movement beneath the pond is primarily vertical downward through unsaturated soil. If runoff to the pond exceeds the infiltration, the water depth in the pond increases as the wetting front continues to move down. Although the soil between the wetting front and the pond bottom is wet, it is not totally saturated due to entrapped air. After the wetting front reaches the water table, the vertical infiltration adds water to the water table aquifer. At this time groundwater movement is primarily horizontal within the saturated aquifer while the water table begins to mound and saturate the soil beneath the pond. If infiltration continues, the mound rises to and above the pond bottom. Once the mound reaches the pond bottom, the area beneath the bottom is fully saturated and the direction of flow is primarily horizontal. See Figure 5-1.

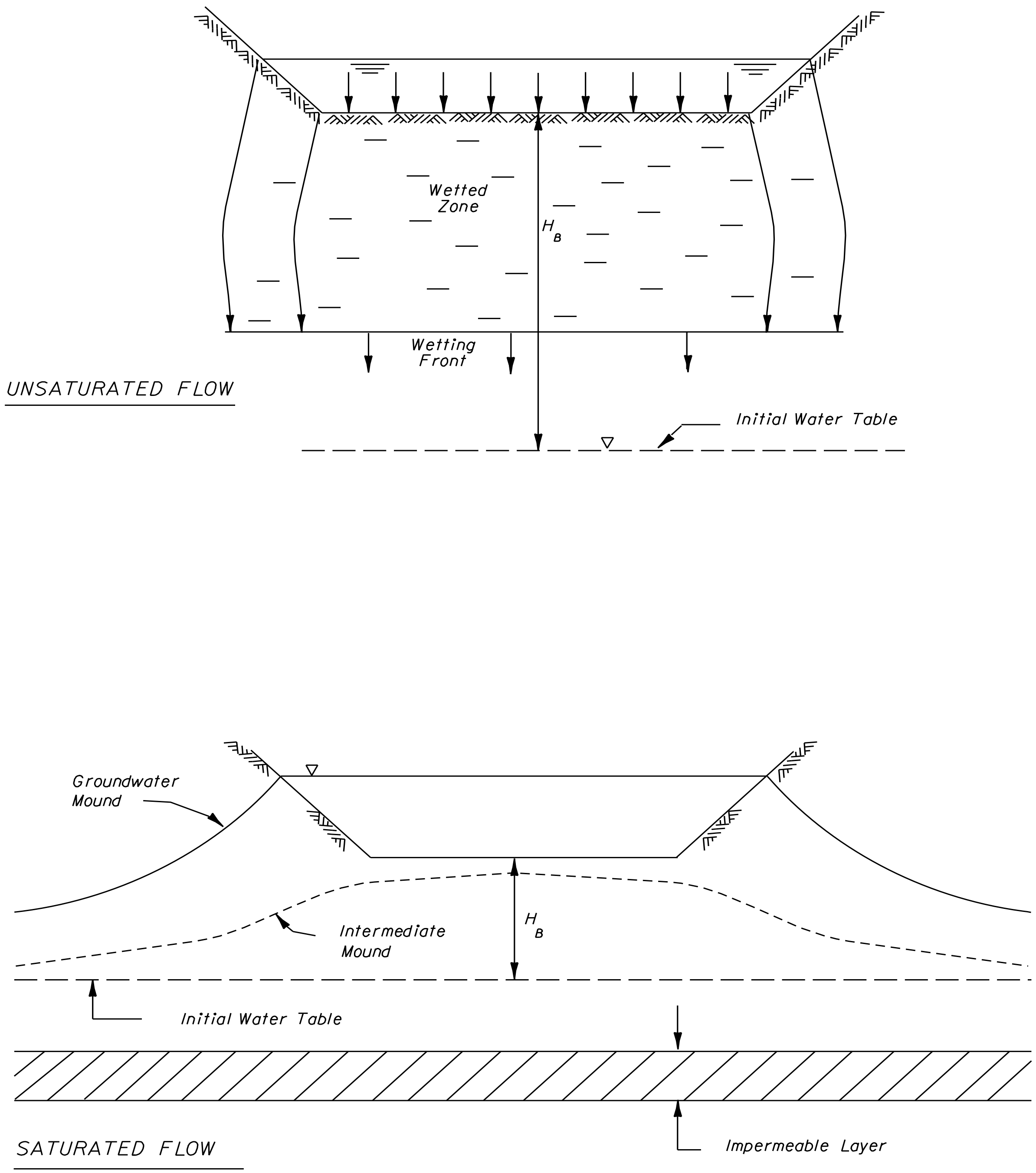


FIGURE 5-1

Groundwater Flow Characteristics During Infiltration

Determining the drawdown characteristics and the recovery time may involve modeling the downward vertical flow through unsaturated soil, or the horizontal saturated flow of the groundwater mound, or both.

5.6.1.1 Unsaturated Flow

The design infiltration rate is: $$I_D = \frac{K_{VU}}{FS}$$

The time necessary to saturate the soil below the pond is: $$T = \frac{f\ H_B}{I_D}$$

The above equations were developed from the modified Green and Ampt infiltration equation. Their derivation is presented in reference a) on page 65.

The total volume of water required to saturate the voids in the soil below the pond bottom is: $VOL_{VOIDS} = A_{PB} \cdot H_B \cdot f$

where
H_B = height of pond bottom above groundwater. See Figure 5-1.
I_D = design infiltration rate
K_{VU} = unsaturated vertical hydraulic conductivity. This is typically obtained from a Double Ring Infiltration (DRI) test. Although infiltration is occurring during the test, the soil is not fully saturated due to entrapped air. The unsaturated K is less than the saturated K. Unsaturated K ranges from ½ to 2/3 saturated K (Bouwer 1978, ASTM D 5126, & Jammal and Assoc. 1989)
f = fillable porosity. See description on page 69.
A_{PB} = area of pond bottom
FS = factor of safety, usually 2.0.

This factor of safety is used to account for the variability of the measurements and for the sediment that will inevitably enter the pond and clog the bottom surface.

5.6.1.2 Saturated Flow

In most of the areas of the state, except the high sandy ridges, the groundwater mound will likely rise to the pond bottom, forcing saturated horizontal flow of the ground water. The most common approach to analyzing saturated flow conditions is to assume flow to be purely horizontal and uniformly distributed across the thickness of the receiving aquifer. The aquifer is modeled as having a single homogeneous layer of uniform thickness and a horizontal initial water table.

Several computer models are available to analyze this. Most use a form of the USGS program MODFLOW. A simplified approach was developed by Jammal and is discussed in the SJRWMD, *"Applicant's Handbook for Regulation of Stormwater Management Systems."* Regardless of which program or technique is used, four parameters are needed to model saturated flow. These are 1) the thickness of the aquifer, 2) the groundwater table elevation, 3) the filllable porosity, and 4) the horizontal saturated hydraulic conductivity of the aquifer.

- Thickness (or Elevation of the Base) of Mobilized Aquifer:

  This is the thickness of soil through which the horizontal flow will occur. This is usually taken to the depth to the top of a confining or very dense layer, such as hardpan, that will restrict the downward vertical movement of groundwater. The maximum value used in the analysis should be the lesser of the depth of the soil boring or the width of the pond. (The maximum depth of the mobilized aquifer is about equal to the width of the pond. Bouwer 1978)

- Groundwater Elevation:

  For modeling the recovery of the treatment volume this is usually the seasonal high water table (SHWT) elevation. For modeling the infiltration of a pond discharging to a closed basin, this groundwater elevation should represent the groundwater elevation during an extreme event like the 100 year-10 day. Currently there is no standard procedure for determining this elevation; nevertheless, it could be substantially higher than the SHWT. For example, where the pond is located near the low in the watershed (lake or flood plain at the low), it may be reasonable to use the 100 year lake or floodplain elevation as the extreme event groundwater elevation. Where the pond is located higher in the watershed, the extreme event groundwater elevation may be closer to the SHWT. Use your judgement and handle these situations on a case by case basis.

- Fillable Porosity:

  This is sometimes called specific yield, storage coefficient, effective storage coefficient, or effective porosity. It is the difference between volumetric water content of soil before and after wetting. The total porosity of a soil is the percentage of the total volume of the material occupied by pores or interstices. The fillable porosity is less than the total porosity because some water exists in soils above the water table; therefore, not all of the unsaturated void space is available for filling. In the zone immediately above the groundwater, capillary rise causes the voids to be substantially filled with water. In fine sands the distance saturated due to capillary rise is roughly six inches. Therefore, the fillable porosity varies with the depth to the water table.

  Specific field or laboratory testing is not usually required for determining the fillable porosity. For most calculations associated with fine sands, the fillable porosity will vary from 0.1 to 0.3 (10% to 30%). The SFWMD has produced soil storage curves which can be used to estimate the fillable porosity. For fine sand aquifers, the SJRWMD recommends using a fillable porosity in the range 20 to 30 percent in infiltration calculations. The higher values of fillable porosity will apply to the well-to excessively-drained, hydrologic group "A" fine sands, which are generally deep, and contain less than 5 percent by weight passing the No. 200 (0.074 mm) sieve.

With all other dimensional and aquifer factors the same, the predicted recovery time decreases as the assumed value of fillable porosity increases. Increasing the fillable porosity from 0.2 (20%) to 0.3 (30%) decreases the recovery time by 15% - 30%.

Horizontal Saturated Hydraulic Conductivity of the Aquifer:

Since the assumption is horizontal flow for the saturated analysis, the hydraulic conductivity should be representative in that direction. This should represent the weighted value of the soil above the confining layer. There are numerous techniques for measuring this value and they are briefly described below. The Department recommends applying a safety factor of 1.5 to 2.0 to the measured values. This factor of safety is applied to account for the variability in the elevation of the impermeable layer, measurement of the conductivity and the estimate of fillable porosity.

Cased hole tests:

Generally measure horizontal hydraulic conductivity if casing bottom is below water table during the test. Generally measure vertical hydraulic conductivity if casing bottom is above the water table during the test. The results should be used with caution if the bottom of the casing is near an impermeable or confining layer.

Uncased hole tests:

This also applies to cased holes which use screen, perforated pipe, or rock bottom to maintain borehole shape. These generally measure horizontal hydraulic conductivity K.

Double Ring Infiltration (DRI) Test:

Generally the DRI measures the vertical unsaturated hydraulic conductivity. Although the Department does not encourage the use of the DRI to obtain the weighted horizontal saturated hydraulic conductivity, If it is the only test information you have, the saturated horizontal hydraulic conductivity could be estimated by applying two adjustment factors as follows.

KVS = 1.5 KVU

KHS = 1.5 KVS (conservative SWFWMD guideline)

Pumping tests:

These are generally expensive and should be reserved for highly sensitive projects. They can overestimate hydraulic conductivity if the bore holes extend into a highly permeable layer which is below a confining layer and the proposed pond bottom is above the confining layer.

5.6.1.3 Special Saturated Analysis

If the aquifer cannot be modeled as having the characteristics discussed above, a more complicated fully three-dimensional model with multiple layers such as MODFLOW, may be necessary.

5.6.1.4 Coordination with the Geotechnical Engineer

When requesting the soils investigation, provide the Geotechnical Engineer with the following information.

- Pond location,
- Approximate pond shape (length, width, plan area configuration),
- Estimated pond bottom elevation,
- Your estimate of SHWT,
- The desired functional characteristics of the pond such as: “This pond will be designed to retain a volume of stormwater for flood control purposes. It should infiltrate ½ the retention volume in no more than 7 days and all of the volume in no more than 30 days.”
- The anticipated groundwater flow conditions / analysis you expect to model based on your preliminary review of the soil and groundwater conditions.

The geotechnical engineers need to know this because the soils investigation can vary depending on the groundwater flow condition anticipated during your design conditions.

Anticipated Groundwater Flow Conditions / Analysis	Soil Investigation (1)
Saturated	1) Thickness of mobilized aquifer. 2) Determine SHWT elevation. 3) Determine weighted saturated horizontal hydraulic conductivity of mobilized aquifer.
Unsaturated (Probably limited to high, sandy ridges)	1) Obtain unsaturated vertical hydraulic conductivity at or near pond bottom. 2) Determine SHWT or confirm that SHWT is at least as low as drainage engineer estimated. 3) Confirm that no confining layer exists between pond bottom and SHWT.
Karst Areas	See discussion in this section.
1) Preliminary results of the soil investigation may dictate that a different soil investigation is necessary. For example, the drainage engineer may have estimated sandy conditions down to a deep water table, planned on doing an unsaturated analysis and requested appropriate soil information. Then the initial soil borings could indicate a confining layer close enough to the pond bottom to warrant a saturated analysis.	

If the groundwater elevation is within 0.6 m (2 feet) of the pond bottom, you can assume that horizontal saturated flow will occur. If the groundwater is farther from the pond bottom, you should compare the volume of the voids under the pond to the volume of runoff which must be infiltrated.

For estimating the groundwater flow conditions, the volume to be infiltrated should be the treatment volume for retention systems discharging to open basins and should be the difference between the 100-year 10-day runoff volume for ponds discharging to closed basins. If the volume to be infiltrated is larger than the volume of the voids under the pond, the groundwater mound will rise to the pond bottom, thus forcing saturated horizontal flow.

Karst Areas:

The WMD and DEP have identified known karst areas and usually have special requirements for stormwater facilities in these areas to assure that water quality of the aquifer is maintained. Sink holes can present problems during or after construction thus it is important that you are aware of potential sink hole locations.

Some sink holes can be only a meter or two in diameter, thus making it difficult to identify their potential. Sometimes potential sink holes can be identified in the field by localized depressions in the ground surface. Using Ground Penetrating Radar may be appropriate in some situations, but has a disadvantage in that it does not penetrate clay layers. Work closely with the geotechnical engineer to identify potential sink holes.

As a preventive measure, a permeable geotextile strong enough to span a small opening could be placed several feet below the pond bottom. This would allow small sink holes to develop without requiring any maintenance work. Doing this will add substantial costs and may not be warranted for all facilities in karst areas. The decision to use such a geotextile should be made jointly by the drainage designer and the geotechnical engineer.

Example 5.4 Discharge to Watershed Without Positive Outlet (Closed Basin)

Given:

• Pre-developed Rdwy Pvmt	= 2-10 foot lanes
• Drainage Area: Includes Rdwy R/W & Offsite Draining to Rdwy	=5.26 ha (13.0ac)
• Offsite land use	= Residential lots averaging 0.20 ha (1/2 ac)
• Proposed Typical Section Curb, & SW	=4-lane urban section; Combined Rdwy, width = 22.2m (73 ft.)
• Length of Rdwy within Drainage Area	= 705 m (2313 ft.)
• Treatment Volume	= 498 m^3 (17600 ft^3)
• The maximum allowable pond stage	= 31.69 m (104 ft)

- Offsite runoff draining to the project will be taken through the pond, not bypassed around.
- Project located near McAlpin, Florida, Rolling terrain, approx. 2% grades, Hydrologic Soil Group B,
- A confining or impermeable layer exists at approximately elevation 28.0 m (92 ft.)
- The saturated horizontal hydraulic conductivity was estimated to be 8 ft/day.
- The SHWT was estimated approximately elevation 28.3 m (93 ft.)

Find: Pond size and outlet control structure configuration.

1. Pre-developed runoff:

Time of Concentration	= 21 min (given)
Area & Curve Number:	
Roadway Pvmt:	= 0.43 ha (1.06 ac) @ CN = 98 (6.1m x 705m)
Pervious	= 4.83 ha (11.94 ac) @ CN = 70 (5.26 ha - 0.43 ha)
Proposed Pond Area	= 0.61 ha (1.50 ac) @ CN = 70 (preliminary size)
Total	= 5.87 ha (14.5 ac) @ CN = 72.1

As in example 5.1, the proposed pond is outside the area draining to the roadway, thus the pond area must be added to the other areas.

Also, as in example 5.1, the roadway right of way to be acquired is within the area draining to the roadway. For your project the acquired R/W may be outside, the area draining to the road, thus requiring that the additional R/W be added to the other areas.

2. Post-developed runoff:
   Time of Concentration = 16 min. (given)
   Area and Curve Number:
   Rdwy, Curb, & SW: = 1.57 ha (3.88 ac) @ CN = 98 (22.2m x 705m)
   Pervious = 3.69 ha (9.12 ac) @ CN = 70 (5.26 ha - 1.57 ha)
   Pond: = 0.61 ha (1.50 ac) @ CN = 98
   Total = 5.87 ha (14.5 ac) @ CN = 80.4

3. Determine the rainfall volumes using the IDF curves (for durations less than 1 day) and the precipitation data of Appendix B of the Drainage Manual.

Rainfall Volumes (inches): McAlpin, FL						
	2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
1 hr	2.2	2.7	3.0	3.5	3.8	4.1
2 hr	2.7	3.3	3.7	4.2	4.6	5.1
4 hr	3.1	3.9	4.4	5.0	5.6	6.1
8 hr	3.6	4.6	5.1	5.9	6.6	7.3
1 day	4.4	5.8	6.8	7.8	8.7	9.6
3 day	5.6	7.2	8.3	9.9	11	12.4
7 day	7.0	8.9	10	12	13.4	15
10 day	7.6	9.5	11.2	13.7	15.2	16

For this example, we will use peak shape factor = 323 for all NRCS hydrograph runs.

4. Assumptions:

   a) Unsaturated Vertical Hydraulic Conductivity: A DRI could not be performed because of the depth of the pond bottom. The unsaturated vertical hydraulic conductivity was estimated from the saturated horizontal conductivity (K_{HS} = 8 ft/day)

      8 ft/day ÷ (1.5 · 1.5) = 3.6 ft/day (See discussion of DRI)

      A factor of safety of 2 was applied to both values, thus the modeled values are K_{HS} = 4 ft/day, and K_{VU} = 1.8 ft/day

   b) Groundwater Elevation: The extreme event groundwater elevation is assumed to be 1 m (3 feet) above the SHWT. Then, extreme event groundwater elevation= 29.3 m (96.0 feet).

   c) Fillable Porosity is assumed = 0.1 (10%) , worst case for fine sands

First Round of Iterations

5. Develop a starting size pond:

   Any approach can be taken to develop the starting trial size for the pond, perhaps a preliminary estimate in the pond siting stage, an educated guess, or a guess based on experience from a similar project. The following approach could be used.

Assume the retention volume will be the difference in runoff volume 100 year -10 day. Using the approach of Example 5.1, the volume difference is 1885 M ^{3}, (66588 ft ^{3}) for this example.

Assume a height of the peak stage over the weir crest. For this example we will use 0.3 m (1 ft). With a peak pond stage of 31.7 m (104 ft), this puts the weir crest at approximately 31.4m (103 ft)

Assume a pond bottom elevation, staying several feet above the estimated extreme event groundwater elevation. For this example, we will start 4 feet above the groundwater elevation with a pond bottom of 30.48 m (100 ft) maintaining 4 feet between estimated peak groundwater and pond bottom.

Determine a pond size and shape that will fit the retention volume between the pond bottom and the weir crest. For this example, a pond with a 61 m (200 ft) x 30.5 m (100 ft) bottom and 1:4 side slopes meets these constraints and will be used as a starting size.

6. Calculate the predeveloped discharge rates and volumes, and route the post developed runoff through the pond. The weir width was arbitrarily selected for this iteration. Using a typical routing program which models infiltration during the storm, you should obtain values similar to the following.

> For the first round of iterations for a pond discharging to a closed basin, it is usually sufficient to run the 100-year FDOT 8-hour through 10-day duration storms.

Table 5.4-1

Pond Configuration:

- Pond Bot. Dims= 61.0 m (200 ft) x 30.5 m (100 ft)
- Pond Bot. El. = 30.48 m (100 ft)
- Avg Side Slope = 1:4
- Weir Crest El. = 31.36 m (102.9 ft)
- Weir Width = 1.52 m (5 ft)
- Volume below Weir Crest = 1942 m^3 (68585 ft^3)
- Allowable Stage = 31.7 m (104 ft)

Modeled Soil Conditions:

- Aquifer Base El. = 28.04 m (92 ft)
- Sat Horiz Cond. (k_{HS}) = 4 ft/day
- Water Table El. = 29.25 m (96 ft)
- Fillable Porosity = 0.1 (10%)
- Unsat Vert Cond. (k_{VU}) = 0.55 m/day (1.8 ft/day)

Design Storm		Disch. Volume $m^3 (ft^3) \times 10^3$	Disch. Rate m^3/s (cfs)	Peak Pond Stage m (ft)
FDOT 8hr - 100 year	Pre	6.105 (216)	(29.1)	
	Post	5.162 (182)	(27.3)	31.76 (104.2)
FDOT 24hr - 100 year	Pre	9.144 (323)	(10.4)	
	Post	8.193 (289)	(10.9)	31.61 (103.7)
FDOT 3 day - 100 year	Pre	13.00 (459)	(8.2)	
	Post	11.94 (422)	(8.4)	31.58 (103.6)
FDOT 7 day - 100 year	Pre	16.67 (589)	(6.1)	
	Post	15.37 (543)	(6.2)	31.55 (103.5)
FDOT 10 day - 100 year	Pre	18.09 (639)	(7.5)	
	Post	16.68 (589)	(7.7)	31.55 (103.5)

Quality Control Retention Volume

Recovery Volume Based on 10-day 100-year Delta runoff.	7 Day $m^3 (ft^3)$	30 Day $m^3 (ft^3)$
Vol. Reqd. to be Recovered =	943 (33300)	1885 (66590)
Vol. Recovered (infiltrated) =	866 (30600)	1511 (53400)

All the post developed discharge volumes are substantially less than the pre-developed discharge volumes of corresponding duration, so the pond retains more than needed. That is, the post developed discharge volumes could be increased. This is done by lowering the weir. Although most of the post developed discharge rates exceed the pre-developed rates, they are close to the pre-developed rates. To maintain similar post developed rates, we will need to reduce the weir width as it is lowered. After making several iterations of weir adjustments the following configuration produces the results in the following table.

For this example, we will add the 1,2, & 4-hour duration storms at this time.

Table 5.4-2
Pond Configuration:
Pond Bot. Dims = 61.0 m (200 ft) x 30.5 m (100 ft)
Pond Bot. El. = 30.48 m (100 ft)
Avg Side Slope = 1: 4
Weir Crest El. = 30.95 m (101.5 ft)
Weir Width = 0.46 m (1.5 ft)
Volume below Weir Crest = 928 m^3 (32768 ft^3)
Allowable Stage = 31.7 m (104 ft)

Modeled Soil Conditions:
Aquifer Base El. = 28.04 m (92 ft)
Sat Horiz Cond.(K_{HS}) = 4 ft/day
Water Table El. = 29.25 m (96 ft)
Fillable Porosity = 0.1(10%)
Unsat Vert Cond. (K_{VU}) = 0.55 m/day (1.8 ft/day)

Design Storm		Disch. Volume m^3 (ft^3) x 10^3	Disch. Rate m^3/s (cfs)	Peak Pond Stage m (ft)
FDOT 1hr - 100 year	Pre Post	2.358 (83.3) 1.822 (64.3)	0.96 (33.8) 0.47 (16.8)	31.55 (103.5)
FDOT 2hr - 100 year	Pre Post	3.456 (122) 3.098 (109)	0.88 (31.1) 0.56 (19.9)	31.62 (103.7)
FDOT 4hr - 100 year	Pre Post	4.638 (164) 4.392 (155)	0.72 (25.4) 0.64 (22.6)	31.67 (103.9)
FDOT 8hr - 100 year	Pre Post	6.133 (217) 5.996 (212)	0.83 (29.1) 0.68 (24.0)	31.70 (104.0)
FDOT 24hr - 100 year	Pre Post	9.152 (323) 9.119 (322)	0.29 (10.5) 0.29 (10.2)	31.40 (103.0)
FDOT 3 day - 100 year	Pre Post	13.00 (459) 12.96 (458)	0.23 (8.2) 0.23 (8.3)	31.35 (102.8)
FDOT 7 day - 100 year	Pre Post	16.65 (588) 16.45 (581)	0.17 (6.1) 0.17 (6.2)	31.28 (102.6)
FDOT 10 day - 100 year	Pre Post	18.08 (638) 17.86 (631)	0.21 (7.6) 0.22 (7.7)	31.33 (102.8)

Retention Volume		
Total recovered in 17 days	7 Day m^3 (ft^3)	30 Day m^3 (ft^3)
Vol. Reqd. to be Recovered = Vol. Recovered (infiltrated) =	464 (16390) 685 (24200)	928 (32770) 928 (32770)

This pond configuration meets the drawdown and discharge volume requirements. The rate requirements are close to being met as the 3 thru 10-day storms are only 0.1 cfs above the pre-developed discharge rates.

Second Round of Iterations

7. Adjust the drainage basin characteristics due to the pond size being smaller than estimated in Step 1. Remember that for this example the pond is located outside the area draining to the road, so changing the pond size also changes the total area. In Step 2, we assumed the entire pond area had a CN = 98. A more refined estimate of the pond area curve number can be made at this time.

   Pond Area:

Water Surf Dims at Peak Stage = 70.7 m x 40.2 m
Water Surface Area at Peak Stage = 0.28 ha (0.70 ac)
Total pond area (incl maint berms & slopes) = 0.45 ha (1.1 ac)
Grassed area within total pond area = 0.17 ha (0.40 ac)

Total Project Area and CN:

Pre-developed Area & Curve Number:
Roadway Pvmt: = 0.43 ha (1.06 ac) @ CN = 98 (from Step 1)
Pervious = 4.83 ha (11.94 ac) @ CN = 70 (from Step 1)
Proposed Pond Area = 0.45 ha (1.11 ac) @ CN = 70
Total = 5.71 ha (14.1 ac) @ CN = 72.1

Post-developed Area and Curve Number:
Rdwy, Curb, & SW: = 1.57 ha (3.88 ac) @ CN = 98 (from Step 2)
Pervious = 3.86 ha (9.54 ac) @ CN = 70(3.69 ha (Step 2) + 0.17 ha)
Pond: = 0.28 ha (0.70 ac) @ CN = 100
Total = 5.71 ha (14.1 ac) @ CN = 79.2

8. Calculate the pre-developed discharge rates and volumes and route the post developed runoff through the pond. Using the same pond / weir configuration as in the previous table produces the following results.

Table 5.4-3
<u>Pond Configuration</u>:
Pond Bot. Dims = 61.0 m (200 ft) x 30.5 m (100 ft)
Pond Bot. El. = 30.48 m (100 ft)
Avg Side Slope = 1: 4
Weir Crest El. = 30.95 m (101.5 ft)
Weir Width = 0.46 m (1.5 ft)
Volume below Weir Crest = 928 m^3 (32768 ft^3)
Allowable Stage = 31.7 m (104 ft)

<u>Modeled Soil Conditions:</u>
Aquifer Base El. = 28.04 m (92 ft)
Sat Horiz Cond.(K_{HS}) = 4 ft/day
Water Table El. = 29.25 m (96 ft)
Fillable Porosity = 0.1(10%)
Unsat Vert Cond. (K_{VU}) =0.55 m/day (1.8 ft/day)

Design Storm		Disch. Volume m^3 (ft^3) x 10^3	Disch. Rate m^3/s (cfs)	Peak Pond Stage m (ft)
FDOT 1hr - 100 year	Pre	2.299 (81.2)	0.93 (32.9)	31.48 (103.3)
	Post	1.586 (56.0)	0.41 (14.5)	
FDOT 2hr - 100 year	Pre	3.369 (119)	0.86 (30.3)	31.58 (103.6)
	Post	2.794 (98.7)	0.51 (17.9)	
FDOT 4hr - 100 year	Pre	4.530 (160)	0.70 (24.7)	31.64 (103.8)
	Post	4.020 (142)	0.60 (21.1)	
FDOT 8hr - 100 year	Pre	5.974 (211)	0.80 (28.4)	31.67 (103.9)
	Post	5.578 (197)	0.63 (22.1)	
FDOT 24hr - 100 year	Pre	8.919 (315)	0.29 (10.1)	31.40 (103.0)
	Post	8.579 (303)	0.27 (9.7)	
FDOT 3 day - 100 year	Pre	12.66 (447)	0.22 (7.9)	31.35 (102.8)
	Post	12.32 (435)	0.23 (8.0)	
FDOT 7 day - 100 year	Pre	16.19 (572)	0.17 (5.9)	31.28 (102.6)
	Post	15.69 (554)	0.17 (6.0)	
FDOT 10 day - 100 year	Pre	17.58 (621)	0.21 (7.3)	31.33 (102.8)
	Post	17.04 (602)	0.21 (7.4)	

Retention Volume		
Total recovered in 17 days	7 Day m^3 (ft^3)	30 Day m^3 (ft^3)
Vol. Reqd. to be Recovered =	464 (16390)	928 (32770)
Vol. Recovered (infiltrated) =	685 (24200)	928 (32770)

This essentially meets all the requirements. The 24-hour and 3-day are critical durations for discharge volume. The 8-hour duration creates the highest stage. The 3-day thru 7-day are critical durations for discharge rate and they exceed the pre-developed discharge rates by less than 2%. This may be acceptable. For this example, several more iterations could be made to bring these rates down without increasing the pond size.

Notice that the retention volume recovered in 7 days was more than necessary and the total volume was recovered in only 17 days. This indicates that we can lower the pond bottom. We can lower the weir crest the same amount that the pond bottom is lowered and maintain similar discharge volumes, which we need to do. As we lower the weir crest, we can reduce the weir width to reduce the discharge rate, which is the primary intent. So after several iterations, the following configuration using two weirs seems to do the trick. Notice it involves a compound weir.

Table 5.4-4

<u>Pond Configuration:</u>
Pond Bot. Dims = 58.5 m (192 ft) x 28.0 m (92 ft)
Pond Bot. El. = 30.17 m (99 ft)
Avg Side Slope = 1: 4
#1 Weir Crest El. = 30.63 m (100.5 ft)
#1 Weir Width = 0.15 m (0.5 ft)
Volume below #1 Weir Crest = 824 m^3 (29120 ft^3)
#2 Weir Crest El. = 35.20 m (103.3 ft)
#2 Weir Width = 4.1 m (12 ft)
Allowable Stage = 31.7 m (104 ft)

<u>Modeled Soil Conditions:</u>
Aquifer Base El. = 28.04 m (92 ft)
Sat Horiz Cond.(K_{HS}) = 4 ft/day
Water Table El. = 29.25 m (96 ft)
Fillable Porosity = 0.1(10%)
Unsat Vert Cond. (K_{VU}) = 0.55 m/day (1.8 ft/day)

Design Storm		Disch. Volume m^3 (ft^3) x 10^3	Disch. Rate m^3/s (cfs)	Peak Pond Stage m (ft)
FDOT 1hr - 100 year	Pre Post	2.299 (81.2) 1.102 (38.9)	0.93 (32.9) 0.23 (8.2)	31.40 (103.0)
FDOT 2hr - 100 year	Pre Post	3.369 (119) 2.223 (78.5)	0.86 (30.3) 0.39 (13.8)	31.55 (103.5)
FDOT 4hr - 100 year	Pre Post	4.530 (160) 3.504 (124)	0.70 (24.7) 0.60 (21.1)	31.61 (103.7)
FDOT 8hr - 100 year	Pre Post	5.974 (211) 5.237 (185)	0.80 (28.4) 0.61 (21.7)	31.61 (103.7)
FDOT 24hr - 100 year	Pre Post	8.919 (315) 8.471 (299)	0.29 (10.1) 0.23 (8.1)	31.41 (103.0)
FDOT 3 day - 100 year	Pre Post	12.66 (447) 12.33 (436)	0.22 (7.9) 0.21 (7.4)	31.36 (102.9)
FDOT 7 day - 100 year	Pre Post	16.19 (572) 15.74 (556)	0.17 (5.9) 0.17 (5.9)	31.27 (102.6)
FDOT 10 day - 100 year	Pre Post	17.58 (621) 17.26 (610)	0.21 (7.3) 0.21 (7.3)	31.36 (102.9)
Quantity Control Retention Volume				
Total recovered in 28 days		7 Day m^3 (ft^3)		30 Day m^3 (ft^3)
Vol. Reqd. to be Recovered = Vol. Recovered (infiltrated) =		412 (14560) 498 (17590)		824 (29120) 824 (29120)

This configuration meets all the requirements for the storms modeled. The 7 and 10-day durations are critical for discharge rate. The 3 and 10-day durations are critical for discharge volume, and the 4 and 8-hour durations create the highest stage. The total retention volume is recovered in 28 days, just under the 30 day requirement. Although it appears that the pond size could be reduced slightly, remember that the earthwork tolerance will slightly effect characteristics of this pond. A slightly lower pond bottom will reduce the aquifer thickness, thus reducing the recovery time. A slightly higher pond bottom will reduce the retention volume and increase the discharge. So when considering the construction tolerance, this configuration looks good.

9. Run the other design storms.

   The other storm frequencies should be calculated to check that the pre-developed discharges are not exceeded. The results are in Table 5.4-5.

10. The stage –storage values used in this example have been based on length and width dimensions applied to a frustum of a pyramid. When you apply the radii to the corners, the storage would be reduced using the same pond dimensions, so use an equivalent stage-area relationship when working with the contours within a Microstation file. Doing so will allow you to configure the pond for aesthetic purposes while maintaining the necessary stage / storage relationship.

TABLE 5.4-5 (Example 5.4, Closed basin)

Same Config. as in Table 5.4-4	100 - year		50 - year		25 - year	
	Disch. Vol. m^3/s (cfs) x 10^3	Disch. Rate m^3/s (cfs)	Disch. Vol. m^3/s (cfs) x 10^3	Disch. Rate m^3/s (cfs)	Disch. Vol. m^3/s (cfs) x 10^3	Disch. Rate m^3/s (cfs)
1-hour Pre Post	2.299 (81.2) 1.102 (38.9)	0.93 (32.9) 0.23 (8.2)	(70.6) (31.5)	(28.7) (6.7)	(60.4) (24.5)	(24.7) (5.2)
2-hour Pre Post	3.369 (119) 2.223 (78.5)	0.86 (30.3) 0.39 (13.8)	(100) (60.7)	(25.2) (9.1)	(84.8) (48.3)	(21.3) (7.3)
4-hour Pre Post	4.530 (160) 3.504 (124)	0.70 (24.7) 0.60 (21.1)	(139) (103)	(21.7) (16.4)	(115) (78.9)	(18.1) (9.9)
8 -hour Pre Post	5.974 (211) 5.237 (185)	0.80 (28.4) 0.61 (21.7)	(181) (155)	(24.4) (14.9)	(151) (125)	(20.4) (9.8)
24 –hour Pre Post	8.919 (315) 8.471 (299)	0.29 (10.1) 0.23 (8.1)	(273) (258)	(8.8) (7.0)	(233) (217)	(7.5) (5.8)
3-day Pre Post	12.66 (447) 12.33 (436)	0.22 (7.9) 0.21 (7.4)	(380) (369)	(6.9) (6.4)	(329) (317)	(6.1) (5.6)
7 day Pre Post	16.19 (572) 15.74 (556)	0.17 (5.9) 0.17 (5.9)	(495) (478)	(5.2) (5.2)	(427) (411)	(4.6) (4.6)
10 day Pre Post	17.58 (621) 17.26 (610)	0.21 (7.3) 0.21 (7.3)	(582) (570)	(6.9) (6.9)	(509) (497)	(6.2) (6.2)
	10 - year		5 - year		2- year	
1-hour Pre Post	(44.6) (14.1)	(18.3) (3.0)	(35.8) (8.9)	(14.8) (1.9)	(22.8) (2.4)	(9.4) (0.6)
2-hour Pre Post	(67.2) (33.9)	(16.6) (5.2)	(53.9) (23.5)	(13.1) (3.7)	(35.8) (10.2)	(8.4) (1.7)
4-hour Pre Post	(92.1) (58.3)	(14.6) (7.3)	(74.1) (42.3)	(11.9) (5.4)	(47.6) (19.8)	(7.7) (2.6)
8 -hour Pre Post	(119) (93.0)	(16.1) (7.1)	(100) (73.9)	(13.4) (5.5)	(63.8) (38.6)	(8.5) (2.7)
24 –hour Pre Post	(189) (173)	(6.1) (4.6)	(147) (130)	(4.7) (3.5)	(92.2) (72.9)	(2.9) (2.0)
3-day Pre Post	(255) (242)	(4.9) (4.5)	(207) (193)	(4.1) (3.7)	(139) (123)	(2.9) (2.6)
7 day Pre Post	(333) (316)	(3.8) (3.8)	(283) (265)	(3.3) (3.3)	(198) (177)	(2.4) (2.4)
10 day Pre Post	(389) (376)	(4.9) (4.9)	(310) (296)	(4.0) (4.0)	(224) (208)	(3.0) (3.0)

Chapter 6

Outlet Control Structures

6.1 Weirs

The most common form of flow control is a weir notched into the side of concrete structure. To maximize the predictability of the flow, the weir should be smaller than the distance between the inside edge of the walls. This is to allow air to get under the nappe. Using a weir size equal to the inside edges of the walls would create an unstable condition when the flow is attempting to spring free from the leading edge of the weir.

Sometimes outlet control structures contain multiple (or staged) weirs such as a small weir at a low elevation with a larger weir at a higher elevation. These compound weirs can be handled one of two ways. SWFWMD recommends treating the lower slot as an orifice, with head (H) measured to the centroid, once the opening is submerged. The upper portion is then modeled with standard weir formulas and the two flows are added. Alternatively, the lower slot computations can be extended to the water surface. Then the flows from the sides of the upper slot are modeled as a separate weir and the flows added. In either case, a totally smooth transition in the performance curve at the stage of the upper weir crest can not be expected. Some amount of manipulation of the curve should be made to smooth it at the transition.

6.2 Discharge Coefficients

The following coefficients are recommended for the typical concrete box outlet control structure. These values, based on a study by the University of South Florida, are documented in a report titled “Performance and Design Standards for Control Weirs, An Investigation of Discharge Through Slotted Weirs,” March 1993; WPI nos. 0510610, & 0510522. Contact the FDOT Research Center at 850-414-4615 to obtain a copy.

The first two tables apply to control devices formed into the wall of the outlet control structure. As a result, the discharge coefficient is affected by the thickness of the structure wall. The discharge coefficient first rises with increasing head and then remains constant. This behavior is observed for both orifices and weirs and is caused by attachment of the flow at the sides of the opening. The wall thickness of the typical FDOT structure can vary depending if the structure is precast or “cast in place.” Unless you specify “cast in place,” assume that the structure will be precast. The Roadway and Traffic Design Standards specify the wall thickness.

TABLE 6-1		
ORIFICE	Discharge Coefficient, C_D	
Condition of Upstream Edge	H/b<0.6	H/b>0.7
Concrete edge (1)	0.276 (H/b) + 0.491	0.709
90^o Elbow Fitting	0.620 (H/b) + 0.284	0.645
(1) These values account for edge imperfections, chipping, wear, and some amount of bevel.		
C_D is dimensionless, to be used the equation: $Q = C_D A_O (2gH)^{1/2}$ A_O = area of opening H = distance of water surface above orifice center b = thickness of the structure wall		

TABLE 6-2				
RECTANGULAR WEIR	Weir Coefficient, C_W			
	0.25<H/b<2.0(1)		H/b>2.0(1)	
Condition of Upstream Edge	Metric	US Customary	Metric	US Customary
Concrete edge(2)	0.258(H/b) +1.35	0.468(H/b) +2.45	1.91	3.45
(1) A typographical error exists in the original report which shows this value to be 2.5 instead of 2.0. (2) These values account for edge imperfections, chipping, wear, and some amount of bevel.				
C_W is dimensional and calculated from $C_W = (2g)^{½} C_D$ C_W is to be used in the equation: $Q = C_W L H^{1.5}$ L = width of weir H = distance of water surface above the weir crest b = thickness of the structure wall				

Thin plate weirs fabricated from metal and bolted over a larger opening in the wall provide a more uniform predictable performance. The metal weir plate should be installed over an opening of sufficient size to ensure that the flow passing over the weir encounters no interference from the headwall. The plate's thickness should be 6 mm (1/4 in) or less to approximate a sharp edge. If constructed as discussed here, the weir coefficient is as follows and is independent of height.

	Metric	US Customary
Weir Coefficient C_W for Thin Plates	1.73	3.13

6.2.1 Submerged Control Devices

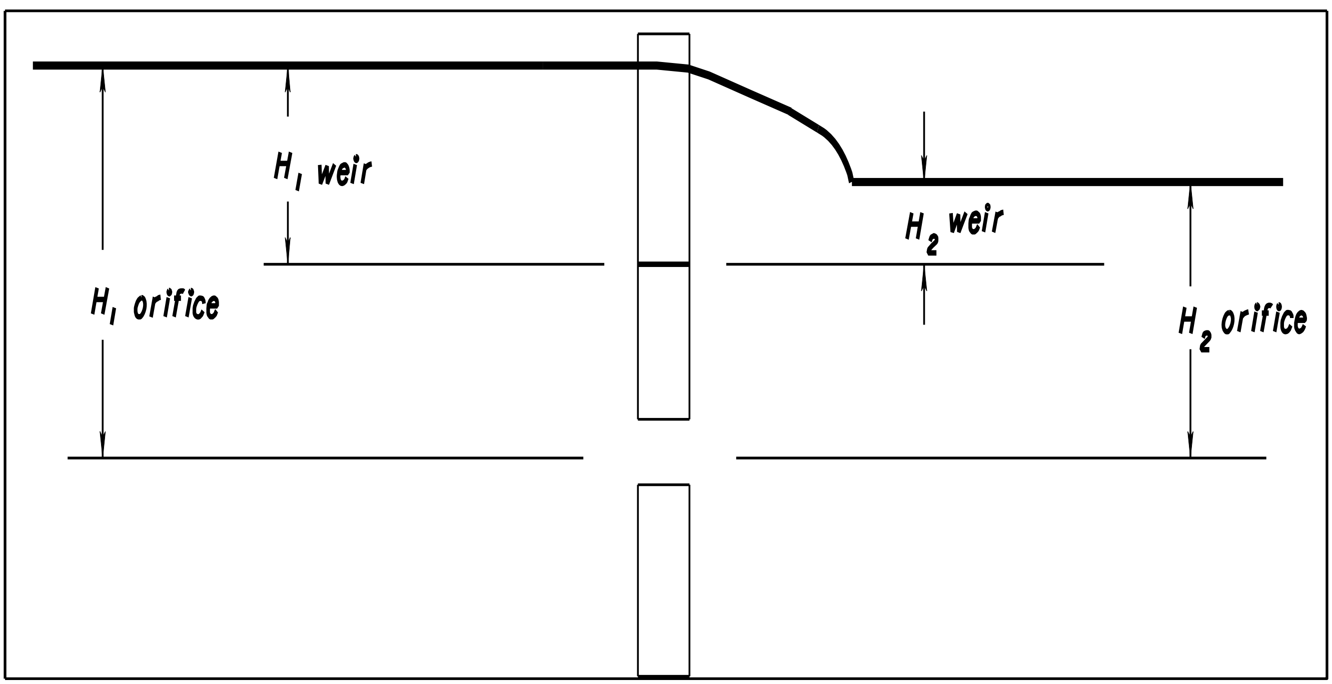
For weirs, use the Villemonte relationship to compute the ratio of flow under submerged conditions to flow under free discharge.

$$\frac{Q_S}{Q_F} = (1 - S^n)^{0.385}$$

where Q_S = flow under submerged conditions
Q_F = flow under free discharge
S = H_2/H_1 = Submergence ratio
H_1 = Upstream headwater
H_2 = Downstream headwater
n = 1.5 for rectangular weirs, & 2.5 for triangular weirs.

Use the following similar relationship for orifices.

$$\frac{Q_S}{Q_F} = (1 - S)^{0.5}$$



6.3 Skimmers

Skimmers are commonly required by the regulatory agency to prevent oil and grease from leaving the pond. The head loss due to such skimmers is minimized if the flow area under the skimmer is three times larger than the flow area of the weir. If this area is provided, you need not calculate the head loss associated with the skimmer.

If it is impossible to provide the flow area mentioned above, the head loss across the skimmer can be calculated using the formula.

$$H_L = k\ V^2/2g$$

where: k = loss coefficient
V = velocity under the skimmer

A loss coefficient, k, of 0.2 is recommended based on a May 25, 1988 SWFWMD Technical Memorandum by R.E. Benson Jr., P.E. Ph.D.

6.4 Miscellaneous

To minimize plant growth, construct a concrete apron around the outlet control structure. It should extend 1.5 meters from the structure.

In wet detention facilities, the outlet control structure generally includes a drawdown device such as an orifice, or a vee notch weir to establish the normal water level and to slowly release the treatment volume. If the drawdown device is smaller than 3 inches wide or less than 20 degrees for "V" notches, include a device to eliminate clogging. Examples of such devices include baffles, grates, screens, and pipe elbows.

It is not necessary to use the ditch bottom inlet type grates on outlet control structures unless needed for safety. If the structure is accessible to the public or will be traversed by maintenance vehicles, grates are recommended.

Always consider the effects of storms which are more severe than designed for. Sometime an overflow spillway is built into the berm. Or additional flow can sometimes pass through the top of the outlet control structure while using the freeboard to store more volume and create additional head.

Appendix A

Rationale for Stormwater Rule Standards

The following is an excerpt from a paper titled "The Evolution of Florida's Stormwater / Watershed Management Program" by Eric H. Livingston, FDEP.

The overriding standards of the Stormwater Rule are the state's water quality standards and appropriate regulations established in other FDEP rules. Therefore, an application for a stormwater discharge permit must provide reasonable assurance that stormwater discharges will not violate state water quality standards. Because of the potential number of discharge facilities and the difficulties of determining the impact of any facility on a waterbody or the latter's assimilative capacity, the Department decided that the Stormwater Rule should be based on design and performance standards.

The performance standards established a technology-based effluent limitation against which an applicant can measure the proposed treatment system. Compliance with the rule's design criteria created a presumption that the desired performance standards would be met which, in turn, provided a rebuttable presumption that water quality standards would be met. If an applicant wanted to use Best Management Practices (BMPs) other than those described in the rule, then a demonstration must be made that the BMP provides treatment that achieves the desired pollutant removal performance standard. The actual design and performance standards are based on a number of factors which will subsequently be discussed.

1. Stormwater Management Goals - Stormwater management has multiple objectives including water quality protection, flood protection (volume, peak discharge rate), erosion and sediment control, water conservation and reuse, aesthetics and recreation. The basic goal for new development is to assure that the post-development peak discharge rate, volume, timing and pollutant load does not exceed pre- development levels. However, BMPs are not 100% effective in removing stormwater pollutants while site variations can also make this goal unachievable at times. Therefore, for the purposes of stormwater regulatory programs, the Department (water quality) and the state's regional Water Management Districts (flood control) have established performance standards based on risk analysis and implementation feasibility.

2. Rainfall Characteristics - An analysis of long term rainfall records was undertaken to determine statistical distribution of various rainfall characteristics such as storm intensity and duration, precipitation volume, time between storms, etc. It was found that nearly 90% of a year's storm events occurring anywhere in Florida produce a total of 2.54 cm (1 inch) of rainfall or less. Also, 75% of the total annual volume of rain falls in storms of 2.54 cm or less. Finally, the average inter-event time between storms is approximately 80 hours (5).

3. Runoff Pollutant Loads - The first flush of pollutants refers to the higher concentrations of storm water pollutants that characteristically occur during the early part of the storm with concentrations decaying as the runoff continues. Concentration peaks and decay functions vary from site to site depending on land use, the pollutants of interest, and the characteristics of the drainage basin.

Florida studies (6, 7) indicated that for a variety of land uses the first 1.27 cm (.5 inch) of runoff contained 80-95 percent of the total annual loading of most stormwater pollutants. However, first flush effects generally diminish as the size of the drainage basin increases and the percent impervious area decreases because of the unequal distribution of rainfall over the watershed and the additive phasing of inflows from numerous small drainages in the larger watershed. In fact, as the drainage area increases in size above 40 ha (100 ac) the annual pollutant load carried in the first flush drops below 80% because of the diminishing first flush effect.

4. BMP Efficiency and Cost Data - Numerous studies conducted in Florida during the Section 208 program generated information about the pollutant removal effectiveness of various BMPs and the costs of BMP construction and operation. Analysis of this information revealed that the cost of treatment increased exponentially after “secondary treatment” (removal of 80% of the annual load) (8).

   Selection of Minimum Treatment Levels - After review and analysis of the above information, and after extensive public participation, the Department set a stormwater treatment objective of removing at least 80% of the average annual pollutant load for stormwater discharges to Class III (fishable/swimmable) waters. A 95% removable level was set for storm water discharges to sensitive waters such as potable supply waters (Class I), shellfish harvesting waters (Class II) and Outstanding Florida Waters. The Department believed that these treatment levels would protect beneficial users and thereby establish a relationship between the rule’s BMP performance standards and water quality standards.

References:

5. Wanielista, M.P., et. al. Precipitation, Inter-event Dry Periods, and Reuse Design Curves for Selected Area of Florida. Final report submitted to Florida Department of Environmental Regulation, 1991.

6. Wanielista, M.P., et. al. Stormwater Management Practices Evaluations. Reports submitted to East Central Florida Regional Planning Council, 1977.

7. Miller, R.A. Percentage Entrainment of Constituent Loads in Urban Runoff, South Florida, USGS WRI Report 84-4329, 1985.

8. Wanielista, M.P., et. al. Stormwater Management Manual. Prepared for Florida Department of Environmental Regulation, 1982.